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No. 853.

STREET GRADES AND CROSS-SECTIONS IN ASPHALT AND CEMENT.

By Robert P. Woods, Jun. Am. Soc. C. E.

Presented March 1st, 1899.

WITH DISCUSSION.

Certain rules and practices in street improvements have been formulated in the larger cities, which, although desirable, cannot always be applied to similar work in the smaller ones. The size, topography, and general local conditions of a city largely determine the character of the street grades and cross-sections, and their plan of arrangement.

A slight departure from customary methods was made by the author in arranging the grades and cross-sections to be used in the recent construction of sheet-asphalt roadways and cement walks, curbs and gutters, in a part of the business district of Wabash, Ind., a small city, having a population of about 11 000. The old grades, as established, were poorly adapted for the proposed improvement, and a careful adjustment of those involved was prepared, with the object of having the finished work as substantial, convenient for use, and harmonious as the local circumstances would permit. Generally, the improvement might be regarded as unnecessary side-hill construction on a comparatively flat surface; a condition unwarranted, in view

of the original topography of the included territory, and yet not now admitting of radical changes.

The erection of many of the buildings within the limits of the improvement had preceded the adoption of any grades by the city authorities, and to this tardiness of the proper municipal officers in establishing common gradients, may be attributed much of the irregularity in the elevations of the floors of the different business rooms.

In the rearrangement of grades, the sidewalks were raised from 1 in. to 12 ins. above the floors of some dozen adjoining store rooms. The owners of the buildings affected, in several instances, waived all claims for damages occasioned by such changes, and the remaining cases were referred to the Board of City Commissioners for assessment of benefits and damages.

There are few places in the improvement where the two opposite walks on any street are on the same grade. This inequality between the elevations of opposite points on the property lines of any street attains a maximum of 2.45 ft. on a street width of 90 ft., and 1.65 ft. on a street width of 66 ft.

The prominent feature in the arrangement of the grades and sections described in this paper is what is here called the "double curb," which consisted of a lower or roadway section and an upper or sidewalk section, so shaped as to form steps from the gutter to the walk. The principal factors entering into the details of the adjustment were the determination of the location of the crown line of the roadway, the height of the exposed vertical face of the curb, and the form and amount of transverse slope of the roadway and walk.

The streets (Fig. 1) are either 90 ft. or 66 ft. wide between outside or property lines, and are paved for the full width with asphalt or cement, with the exception of one block on Market street (not shown in Fig. 1), where half the sidewalk space on either side of the roadway is paved with cement and the other half graded into lawn. On the 90-ft. street, the roadway between curb faces is 59 ft., and either walk, including the curb, is $15\frac{1}{2}$ ft. wide. The 66-ft. streets have roadways 40 ft. in width between curb faces, and either walk, including the curb, is 13 ft. wide.

All roadways, exclusive of gutters, are paved with sheet asphalt consisting of a 2-in. top, and a 1-in. bituminous binder, on a 6-in. hydraulic concrete base. The walks, curbs and gutters are laid with granite-faced cement concrete; the walks being 5 ins. and the curbs and gutter 6 ins. thick.

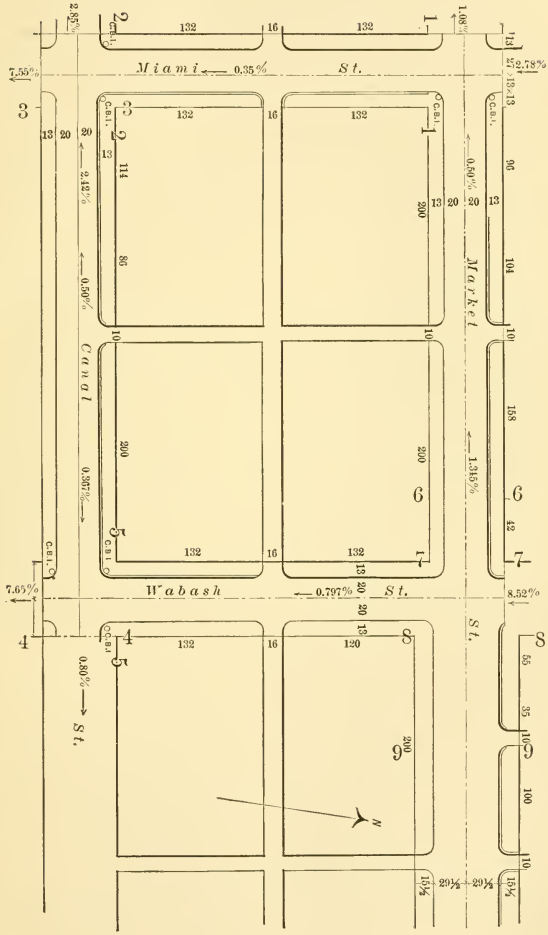

Miami St.
Wabash St.
Canal
Market St.
0.35%
0.797%
0.50%
0.367%
0.80%
2.42%
1.345%
7.55%
7.65%
2.85%
1.08%
2.78%
8.52%
132
16
120
200
114
86
96
104
158
42
55
35
100
10
13
20
29½
15½
N

FIG. 1.

The curb and gutter throughout were combined, and constructed as a monolith, in sections 6 ft. in length. The roadway edge of the curb is turned with a 2-in. radius, and the junction of the curb and gutter was effected with a radius of $1\frac{1}{2}$ ins. From out to out the combined curb and gutter measures 22 ins.; 6 ins. for the curb and 16 ins. for the gutter. The exposed vertical height of the curb varies from a minimum of 0.33 ft. to an extreme of 0.85 ft.

The double curb is composed of a lower or roadway section and an upper or sidewalk section constructed on parallel lines; the roadway face of the latter being 12 ins. from the roadway face of the roadway curb and the top flush with the sidewalk. The exposed vertical height varies from nothing to a maximum of 0.92 ft., and the edge is turned with a 2-in. radius. The lower or roadway section has a top width of 12 ins., and is the regular roadway curb increased by the addition of a 6-in. block of cement fitted into the space between the regular curb and the sidewalk curb, the top surface of the additional piece being flush with the top of the roadway curb. The exposed vertical height of the roadway section varies, as in the case of the regular roadway curb, from 0.33 ft. to 0.85 ft. This double curb was designed to be used where the height from the gutter to the top of the walk was more than 10 ins., but did not exceed 21 ins. Risers of 10 and 11 ins. in steps are rather excessive, but it was thought that the exigencies of the situation permitted their adoption in the few places where they were used.

All cut-off boxes for gas and water supply to buildings, were placed in the sidewalk space immediately back of the regular curb, and where encountered contiguous to the double curb, they were fitted into the widened portion of the lower section.

Several short stretches of good stone sidewalks had been constructed by property owners under previous city ordinances, and after some adjustment were allowed to remain. They comprised the north walk on Canal Street, from Miami Street to Wabash Street, and several small lengths on both sides of Market Street, between Wabash and Huntington Streets. Those on the north side of Market Street had been paved to a width of only 12 ft., thus necessitating a widening of several feet.

Frequently, in street construction, where the side lines are on different gradients, one gutter is placed higher than the other, and the transverse form of the roadway made either on a straight line between the two gutters, or in the form of half an arc, the chord of which is

OUTLINES OF CROSS SECTIONS OF STREETS.

ALL MEASUREMENTS IN FEET.

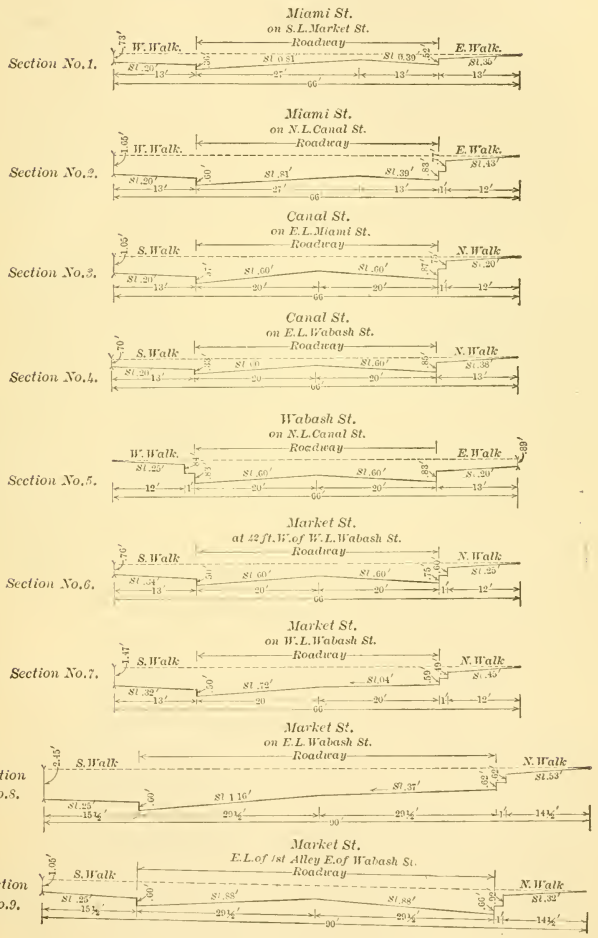


FIG. 2.

double the width of the roadway. When the former is used, sags or depressions have a tendency to develop in the pavement, especially where the roadway is wide, and with either one, the objection might be raised, that carriage and street car travel are not best accommodated. It was decided to preserve a crown line within the roadway throughout this improvement, due consideration having been given to other conditions; and to keep it preferably in the center; but, as an expedient, in one block of pavement on Miami Street, the crown line was shifted 7 ft. from the center toward the high side of the street, as shown in Sections Nos. 1 and 2, Fig. 2. With this single exception the crown line was maintained in the center of each roadway.

The three usual forms for roadway crowns are, the parabolic and circular curves, or a combination of either one with tangent sides. Of these, the first two are nearly always alike, while the last may be arranged quite differently from either. The form used by the author was a parabolic curve, and yet, for all practical purposes, it might be called the segment of a circle, since a calculation of both curves shows their corresponding ordinates to be almost identical where the proportion of rise to span is so small as prevails in the construction of city pavements. The distance from the edge of the gutter to the crown line was divided into eight equal parts, the division points being numbered consecutively from "0" at the crown line, to "8" at the gutter, and the formula, as deduced for determining the ordinates from the surface of the pavement to a horizontal line level with the crown point, is as follows:

Ordinate at the crown point 0 = 0
" " 1 = 0.0156 C or $\frac{1}{64}$ C
" " 2 = 0.0625 C " $\frac{4}{64}$ C
" " 3 = 0.1406 C " $\frac{9}{64}$ C
" " 4 = 0.2500 C " $\frac{16}{64}$ C
" " 5 = 0.3906 C " $\frac{25}{64}$ C
" " 6 = 0.5625 C " $\frac{36}{64}$ C
" " 7 = 0.7656 C " $\frac{49}{64}$ C
" edge of gutter " 8 = 1.0000 C " $\frac{64}{64}$ C

in which C equals the crown, for the distance specified.

The amount of crown of each roadway from the face of the curb to the crown line is 3% of the horizontal distance between the two points named. Of this amount the gutter has a uniform slope of 0.08 ft., in its width of 16 ins., and the remainder is the rise or crown for the

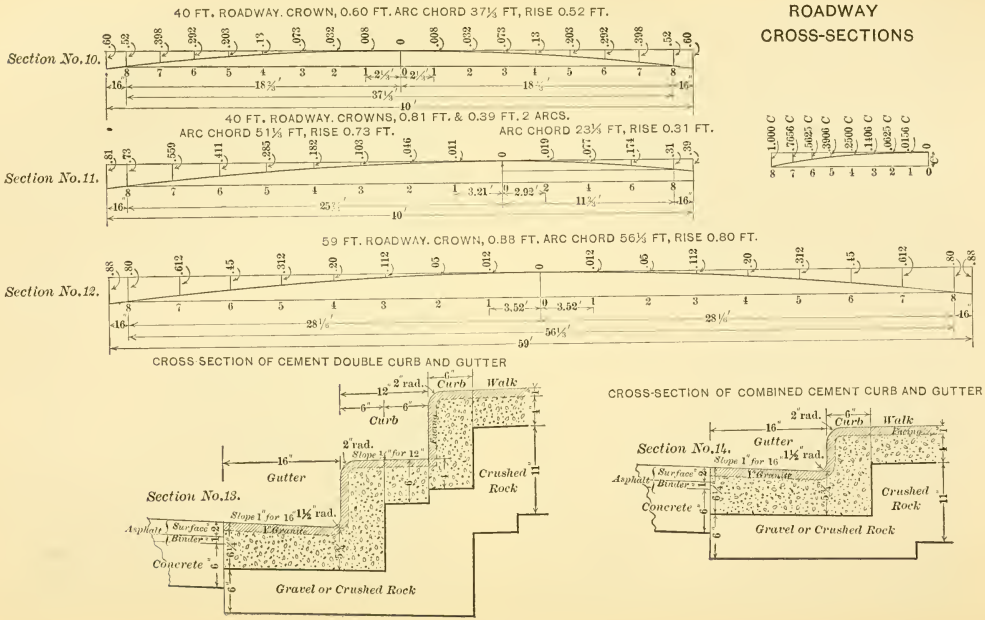

ROADWAY CROSS-SECTIONS
40 FT. ROADWAY. CROWN, 0.60 FT. ARC CHORD 37½ FT, RISE 0.52 FT.
Section No.10.
40 FT. ROADWAY. CROWNS, 0.81 FT. & 0.39 FT. 2 ARCS.
ARC CHORD 51⅛ FT, RISE 0.73 FT.
ARC CHORD 23½ FT, RISE 0.31 FT.
Section No.11.
1.000 C
.7056 C
.5625 C
.3906 C
.2500 C
.1406 C
.0625 C
.0156 C
0 C
59 FT. ROADWAY. CROWN, 0.88 FT. ARC CHORD 56⅛ FT, RISE 0.80 FT.
Section No.12.
CROSS-SECTION OF CEMENT DOUBLE CURB AND GUTTER
Section No.13.
Curb
Walk
Gutter
Crushed Rock
Asphalt
Surface
Binder
Concrete
Granite
Gravel or Crushed Rock
CROSS-SECTION OF COMBINED CEMENT CURB AND GUTTER
Section No.14.

FIG. 3.

asphalt portion. In the case of a roadway 40 ft. in width between curb faces, with the crown line in the center, the total rise or crown is 0.60 ft., and the crown in the asphalt portion which is included between the gutter edges is 0.52 ft. This latter rise, as before stated, is proportioned on the form of a parabolic curve.

The crown and gutter lines in each block of pavement are on parallel longitudinal grades throughout, excepting the small deviations made on Market Street on either side of the Wabash and Market Street intersection. These roadway grades range from a minimum of 0.35% to a maximum of 2.42 per cent.

The cross-slope of the sidewalks is on a uniform inclination from the property lines of the streets to the curbs, and varies in amount from 0.015% to an occasional 4% grade.

The plan, Fig. 1, exhibits the dimensions of the streets, the location of the regular and double curbs and the longitudinal grade of the roadways. The two large similar figures on the same line refer to the numbers of the detailed outline sections shown in Fig. 2.

Outlines of nine different cross-sections of the streets are represented in Fig. 2, indicating the amount of slope given to the roadway and walks, together with the excess of elevation of one walk over the other at the location cited. Sections Nos. 10, 11 and 12, in Fig. 3, are the forms of crown for the several roadways, with the ordinates given in decimals of a foot, as calculated by the parabolic formula. The form for the 40-ft. roadway having the crown line in the center, is shown in section No. 10, the curve corresponding closely with a circular arc having a radius of 335.30 ft. Section No. 11 is the form for the 40-ft. roadway having the crown line shifted 7 ft. from the center, and contains two curves. The short one on the right has a rise or middle ordinate of 0.31 ft., a half chord of $11\frac{2}{3}$ ft., and corresponds closely with a circular arc having a radius of 219.68 ft.; the longer one on the left has a rise or middle ordinate of 0.73 ft., a half chord of $25\frac{2}{3}$ ft., and corresponds with a circular arc having a radius of 451.58 ft. The form shown in section No. 12, was used for the 59-ft. roadway, the corresponding circular arc having a radius of 496.25 ft.

The double curb and gutter, and the combined single curb and gutter are shown in sections Nos. 13 and 14, respectively.

Two views of Wabash Street, taken before and after the construction of the improved pavement, are shown on Plate I.

FIG. 1.

FIG. 2.

DISCUSSION.

JAMES OWEN, M. Am. Soc. C. E.—In regard to the question of crown, the speaker's ideas are somewhat decided. It is his desire to impress upon everybody the fact that the roadway itself is made primarily for wagon travel, and that, *per se*, a level cross-section is much more advantageous for such travel than any other cross-section. A departure therefrom can only be tolerated because there are overcoming disadvantages against it. The original country road was piled up probably 2 ft. in the center so that the water could be shed to the gutters. As the character of highways has been improved, the crown has been gradually decreased, until, with the improved pavements of to-day, it seems to the speaker that the crown is scarcely a matter of consideration, and in his practice, in both asphalt and brick pavements, he has reduced it to about 3 ins. in, say, 40 ft. Mr. Owen.

The only advantage of the crown is to shed the water to the sides in order to avoid the wash of the perishable material of the roadway and the damage ensuing therefrom. For a permanent, smooth roadway, either of granite blocks, brick or asphalt, no argument can be advanced in favor of any crown at all, except such as is needed to shed the water, at the proper point, away from the street entirely. In fact, the question of using the roadway itself, from curb to curb, as a gutter, is now quite often considered; and the question of economy in constructing storm sewers is being considered in that light. For instance, on a side hill, with streets at right angles to the slope, it is economical to use the streets themselves as gutters and discharge the water into the storm sewer at each cross street, the latter being provided with a crown in order to divert the water into the sewer. The speaker thinks that practice is going to be more in vogue than it has been.

In considering the elimination of crown as much as possible from street construction, it will be seen that these difficulties will disappear entirely. The speaker agrees with Mr. Tillson that he could have solved the problem without the crown, and could also have avoided the steps in the gutter line.

The relation between the roadway and the depth of the gutters is purely arbitrary. In the business part of Yonkers, two or three years ago, the speaker noticed that, in the improved streets, the sidewalks were from 18 ins. to 2 ft. above the roadway. That, of course, had come about from the fact that the original grade was being altered, and, in permanent improvements, some kind of a grade had to be established, without relation to the sidewalks. The result was that the sidewalk was adapted to the improvement of the house, and it would have been detrimental to the property to cut the sidewalks down; that

Mr. Owen the sidewalk, being 2 ft. above the street, offered no practical disadvantage, and that wagon travel could be more evenly handled with a sidewalk 18 ins. high than with one only 8 ins. high. A person can step out from a carriage more easily, and can more readily transfer freight to or from a wagon when the sidewalk is 18 ins. high than when its height is only 8 ins., and in this instance the foot travel on the spaces between the cross streets is very small. By making proper arrangements at the street intersections for the accommodation of foot travel passing from the sidewalk to the street, no practical or permanent disadvantage will result in having the sidewalk higher than the stereotyped 8 ins. Furthermore, the saving is a disadvantage, in any way it can be considered, both by stepping from a carriage a larger distance from the sidewalk and also in carrying freight over that space.

The question of the relation of the sidewalk on one side of a street to that on the other is continually arising. The speaker has constructed roadways in which there was a difference of 4 ft. in the elevation of the sidewalks. Of course, that would not be considered an ideal construction, but it had to be used. In such cases a standard curb was adopted, and the crown of the roadway was placed about 6 or 8 ft. from the center toward the higher side.

Mr. Ulrich. D. ULRICH, Assoc. M. Am. Soc. C. E.—In constructing asphalt pavements the speaker's experience has been limited, and has generally been in localities where no curb lines have been established. For that reason he has had no trouble in making the crown suit the width of the road, and has in nearly all cases made it about 4 ins. for a 60-ft. roadway. In constructing macadam roads, however, he has ordinarily made it somewhat higher. For a 20-ft. road the crown was generally made 4 ins.; for a 30-ft. road, 6 ins.; and for a 40-ft. road, 8 ins.

Mr. North. EDWARD P. NORTH, M. Am. Soc. C. E.—In what is now the Borough of Manhattan, in the City of New York, under the old charter, the Water Purveyor had charge of all new paving, repaving and repairs to pavements. New pavements, and all grading, regulating, curbing and flagging, were paid for by assessments on the property benefited. All repaving and repairs to the pavement, except those due to openings in the streets made by plumbers or companies having pipes or conduits under the pavements, were made by the city and paid for out of appropriations.

City ordinances require sidewalks to have a rise of 1 in. per 5 ft. from the curb. The lower part of the city is mostly without official grades, but, commencing about at Houston Street, grades have been established. It has not, however, been established whether such grades refer to the curb stones or the middle of the streets. This uncertainty is, at times, a matter of convenience to the engineer who wishes to improve the profile of a street that is to be repaved, as it allows a comfortable working latitude.

Mr. North.

There is a noticeable exception to this statement, viz., the streets joining the Riverside Drive. Here, the grades were established by the Park Commissioners for the curbs, both on the drive and on the streets back to the house lines. Unfortunately, they were substantially level, although the same authorities had established quite steep grades on the streets (in one instance, though not adjoining the Riverside Drive, a grade of 18%, which is thought to be the maximum). As any departure, in either paving or curb setting, from the established grade, is liable to result in vacating the assessment, these level areas of 18 to 20 ft., joining, say, a 10% rise, have to be retained, and are objectionable.

There is also a troublesome practice by those who are called "skin builders," who, contracting to build a house with certain dimensions between the floors, avoid the excavation of 2 and sometimes 3 ft. of rock, by establishing a curb grade suited to their work and also by increasing materially the slope of the sidewalk. This swindle is generally undetected, particularly on unpaved streets, until the house is paid for, and, in some instances, causes considerable trouble and dissatisfaction to the property owner, as the curb is generally reset on the proper grade when the street is paved, which either necessitates relaying the sidewalk in front of the property, or an approximation to the practice described by the author.

Except as in the instances cited, the engineer in charge of the streets in the City of New York probably has as little trouble with grades as in any other city or town. No ordinance governs the depths of gutters, but they are called for in all streets; nor is any crown specified in the city ordinances.

On taking charge of the paving in what is now the Borough of Manhattan, with the office of Water Purveyor, it was decided to make the crown of all asphalted streets, with curbs at equal elevations, $\frac{1}{100}$ of the distance between curbs, and the normal depths of gutters between $3\frac{1}{2}$ and $4\frac{1}{2}$ ins. The crown adopted, combined with the low curbs, gives a street of apparent civilization and luxury; the high crown being a survival from the pathmasters' "turnpiking" with heavy soils; and the deep gutters antedate sewers. Some of the asphalting companies objected to these unprecedentedly flat transverse profiles, as the cost of thorough maintenance is, doubtless, greater than where water will run off more quickly. It was with some satisfaction that notice was taken that the French engineers had at last adopted the same rise, viz., $\frac{1}{100}$, for the wheelway of the Alexander III Bridge, drawings of which were presented at the meeting of the Society on February 15th, 1899.

No complaint has ever been made, to the speaker's knowledge, of cellars flooding through insufficient capacity of $3\frac{1}{2}$-in. gutters; so it apparently may be assumed that on asphalted streets with 4-in. gutters and sidewalks of one-half the width of the wheelway, rising at the rate

Mr North. above mentioned, no danger to cellars is to be apprehended if the sewers have sufficient capacity to take off the water.

Objections are frequently made to longitudinal grades on asphalted pavements, futile efforts having been maintained for years by one of the city departments to confine asphalt to grades of $2\frac{1}{2}\%$ or less. Small attention, however, is generally given to transverse grades, although every one knows that while on asphalt a horse may stumble in the line of draft, he almost invariably falls at right angles to it. As a matter of fact, it is difficult to get a horse that knows anything about asphalt to travel anywhere except in the center of a high-crowned street in slippery weather. This obvious danger from excessive transverse grades led to the suppression of the crown in streets having a transverse fall of much over 3 per cent.

The first asphalted street paved without a crown was Eightieth Street between Fourth and Madison Avenues. On Fourth Avenue the difference in elevation between curbs was 1.3 ft.; by making a 7-in. curb on the north and 4-in. curb on the south side, the fall was reduced to 1.05 ft., or a grade of $3\frac{1}{2}$ per cent. The foundation was the old trap-block pavement relaid. The transverse profile at Fourth Avenue was a right line; at Madison Avenue it had a rise of $\frac{1}{100}$, and the south curb was somewhat lower than the north curb. The grades were "boned in" for 405 ft. between the two end-profiles on the center, quarter and curb lines. No settlement or distortion has been observed.

The above-mentioned practice was often modified, where the relative heights of the curbs would admit, by reducing the length of the warped surface to 150 or 200 ft. In other instances it was thought advisable to dispense with a crown for one or more blocks. In such cases there was no gutter on the upper side. No instances have come to the speaker's knowledge where sags or depressions in the foundations have developed, either with a relaid stone pavement or with a concrete foundation. In some cases, however, a crown of about 1 in. was worked in, consequent to the turnpike idea, as aforesaid.

Experience in the City of New York is decidedly against concrete curbs. While a good curb can be made of concrete, with proper materials and care, poor work cannot be prevented, and many, if not most, of the concrete curbs laid have been scamp affairs, with concrete made of cinders and plastered with a thin coating of poor cement. The wheel of a grocer's delivery wagon would cut this thin coating off, and backing a coal cart against the curb would break it. The poorest curbs were put down in the tenement house districts where the surroundings are squalid enough without the additional aid afforded by broken curbs. Fortunately, there was a city ordinance requiring stone curbs. Under this ordinance, samples of the curbs were taken to court and their construction, for a time, at least, stopped.

Mr. Owen's suggestion, that the gutter should be in the middle of

the street, is worth consideration, particularly for asphalted streets. Rain would flush the pavement more quickly, and as the transverse profile would be two right lines joined by a concave curve, instead of a convex curve increasing in rate toward the side gutters, as at present, the whole carriage-way on level streets could be made equally available for traffic, as it would not be necessary to have the transverse grade as high as 1 per cent. A single gutter in the center of the street is found in many Spanish towns, and was considered for some of the narrow streets in the lower part of the City of New York, but the ordinance calling for gutters and curbs prevented its use. Mr. North.

G. W. Tillson, M. Am. Soc. C. E.—The speaker has been much interested in this paper, as it discusses a question which he has been studying for some years. The author has been confronted with the problem that always presents itself to every engineer who attempts to pave a street where the grades have been established by one who never expected the street to be paved, or had no idea of the proper cross-section of a paved roadway; or where a side-hill street has been built up, with no consideration for the future improvement of the street, but only according to the whims of the individual property owners. Mr. Tillson.

When such conditions exist, and the entire width of the street is to be improved, the difference in elevation of the curbs must first be reduced to a minimum, by giving the sidewalks on the high side the maximum, and on the low side the minimum, allowable slope. In the same way, by applying the maximum gutter to the high side, and the minimum to the low side, the difference in elevation of the gutters is made as little as possible.

The best cross-section for the roadway itself is now to be considered. Some engineers would make this a straight line from gutter to gutter. The speaker does not think this is right. Any little variation from a true plane, caused either by imperfect work or the action of travel, will show much more plainly than when the surface is slightly curved. It is also better to prevent all the water from running transversely across the street. By locating the crown of the street on the quarter, instead of at the center, and by giving a slight rise from the higher gutter to the crown, these two difficulties are overcome, the difference in elevation is but slightly changed, and the distance in which to overcome it is made considerable.

The author has adopted 0.33 ft. for a minimum and 0.85 ft. for a maximum gutter. These figures agree very closely with the speaker's ideas; but a gutter involving a step of 10 ins. should be used only in very extreme cases, and when to this is added another step of 11 ins., and the two are separated by a tread of 12 ins., the construction is bad, and should be avoided, unless it can be shown to be absolutely the best under the existing conditions. A double curb, as constructed, is expensive, unsightly and dangerous.

Mr. Tillson.

In Fig. 4 the speaker has taken five of the worst sections shown in the paper and has replatted them, and on the same base has platted the sections he would have used. The author's section is shown by a full line and the speaker's by a dotted line, the latter result being obtained by using sidewalk slopes and gutters within the limits recommended by the author.

In Section No. 5 it will be seen that by making gutters 0.66 ft. and 0.36 ft. on the high and low sides, respectively, making the crown on the quarter, and allowing a rise of 0.16 ft. from the gutter to the

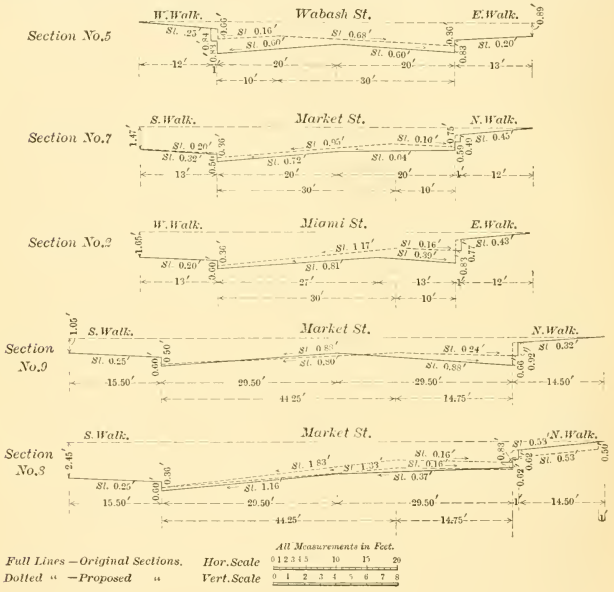


FIG. 4.

quarter, a section can be obtained which in no place has a greater slope than the author's, which has a step of 0.66 ft. instead of 0.82 ft. and which does away entirely with the double curb. On the street, the fact that the two sides are unbalanced would hardly be noticed, except by a trained eye, and for practical use this section is as good as the other.

In Section No. 7 the slope of the sidewalk on the low side is reduced to the minimum and the depth of the gutter made 0.36 ft. On

the other side the maximum gutter is taken, and the rise to the quarter is the same as used in Section No. 5. The resulting section has no slope in excess of the author's, does away with the double curb, and has a curved surface where that shown by the author is practically level, there being but 0.04 ft. fall in 20 ft. Mr. Tillson.

In Section No. 2 the gutter on the low side is made 0.36 ft.; on the high side it is placed at the top of the author's real curb, and the section is figured as in No. 7. This slope is slightly in excess of that used by the author, but not to such an extent as to be detrimental, while on the other hand it does away with the necessity of the double curb.

Section No. 9 is on a street 90 ft. wide. The low gutter is made 0.5 ft. deep, and on the high side it is placed at the top of the curb, as in No. 2. By making the quarter crown 0.24 ft. above the high gutter, a section is obtained with a slope even less than the author's, and with no double curb.

In Section No. 8 the worst conditions are encountered. The two gutters are made 0.36 ft. and 0.83 ft., respectively, and the quarter crown 0.16 ft. This gives as a section the upper dotted line, which is rather bad, but the speaker thinks that even here he would adopt it in preference to the double curb and the section shown by the author, having a slope of 0.37 ft. from the gutter to the center, and thus throwing the entire surface water to the low side of the street.

If, however, it is thought that the slope is too great, the speaker would reduce it by placing the step near the building, rather than at the curb line. The exact distance from the property line must be determined by the conditions. If, as is generally the case, a certain space is allowed to property owners for a display of goods, or for area-ways or courtyards, this step might be located at the edge of this space, with no inconvenience to any one. In this event the sidewalk would be laid on the same slope as before, but 0.5 ft. lower, and the cross-section of the street would be as shown by the lower dotted line, which the speaker thinks is much to be preferred to that used by the author.

If the latter plan were adopted, the upper sidewalk would extend some 3 or 4 ft. beyond the property line at the cross streets, and then return up the side street, running the grade out in a short distance and thus doing away entirely with the two steps at the curb. Should the gutters shown by the author be retained, the actual street would be improved by adopting the section shown by the lower dotted line.

If the above new sections should be used on a street where existing conditions were as described in the paper, no double curb would be required, and, for practical purposes, the cross-sections would be as good as those shown by the author.

Mr. Tillson. These results have been obtained by varying from the author's procedure in two ways: First, by making the crown on the quarter in every case where the difference in elevation of the curbs requires it; second, by not attempting to make the slope on both sides of the center uniform.

The speaker has used the first method for several years with uniformly good results, and is fully satisfied that, under the conditions considered in the paper, it is the proper course to adopt.

While the author has not always attained the results mentioned in the second reason, he has done so in Sections Nos. 5 and 9, and, apparently, has sought to in others, thus necessitating the use of the double curb, or a needlessly high step.

Mr. Hankinson. A. W. Hankinson, Jun. Am. Soc. C. E.—In determining crowns for asphalt pavements in New York City, that is, in the Borough of Manhattan, a standard section is assumed of $4\frac{1}{2}$-in. gutters, with a crown of $\frac{1}{100}$, and a quarter height of $\frac{1}{160}$ of the width of the roadway, thus making the proportion of crown to quarter as 8 : 5. When the curbs are not level, the gutter on the high side is deepened to a maximum of 9 ins., and if the gutters are still of unequal elevation, one-half of the difference of elevation is subtracted from the standard crown, in effect securing the standard fall of 2% from the crown to the gutter on the low side. It will be seen that in a 30-ft. roadway, with a difference of curb level of $11\frac{1}{2}$ ins., the result of following this plan will be a street with no crown at all, the pavement being on a straight grade from gutter to gutter. It seldom occurs that a greater difference in curb elevation is necessary on 30-ft. streets, as in most cases one of the curbs can be raised or the other lowered without doing very much damage to the adjoining property. The wider the roadway, the greater the difference in curb level that can be allowed; as in 60-ft. streets, the crown being $7\frac{1}{2}$ ins., a difference of 18 ins. can readily be adjusted.

The author, having the opportunity of adjusting not only the roadway, but also the sidewalk, it would seem that even his maximum difference of level of 1.65 ft. in 66 ft. could have been accommodated without the use of the stepped curb, by lowering the sidewalk on the high side. A step at the entrance of each store would be less objectionable than a continuous one at the curb for the length of the block.

Roadways of a greater width than 30 ft. have, in New York, usually a street railroad occupying 15 ft. of the street. When such railroad has no underground conduit to be used as a drain, the rails are placed on the contour of the pavement arch, as in a level 60-ft. section, the center rails would be $\frac{1}{2}$ in. below the crown and the outside rails dropped $\frac{3}{4}$ in. from the center rails. As the pavement between the rails is crowned up 1 in., this $\frac{3}{4}$-in. drop amply provides for drainage. In the case of an underground trolley with slotted conduit, the slot

rails are placed on the contour of the pavement crown and the tram rails made level if possible. Should, however, the difference of curb elevation be very great, the track is "terraced," that is, one track, that on the low side of the street, is dropped from 1 to 3 ins. from the other. By terracing the tracks and raising and dropping the outside rails $\frac{3}{4}$ in. on the high and low sides of the street, respectively, a difference of 19 ins. in curb level on a 60-ft. street has been adjusted without draining to one gutter. Mr. Hankinson.

The crowns used by the author give a much steeper cross-grade than could be safely used in New York City. His quarter height being quite high in proportion to the crown, a very sharp incline from quarter to gutter results. It may be that Wabash, Ind., is not subject to the fogs, mists and drizzling rains which make the New York pavements so slippery at times, and that a cross-grade of 3%, entirely unsuited to the climatic conditions obtaining in New York, would be eminently proper in an inland city.

J. M. Evans, Jun. Am. Soc. C. E.—Judging from the speaker's experience in street work in New York City, the double curb would find little or no place of usefulness there; the objections already mentioned in the discussion being sufficient to exclude it. One objection, however, which has not been mentioned and yet is of grave importance, is the impossibility of keeping such a step in a sanitary condition, as it would afford a convenient repository for filth. In New York City it is the custom to place hydrants, lamp-posts, fire-alarm posts, etc., very near the curb. In the double-curb arrangement these would naturally be set in the tread of the step, and the space around them would soon become filthy and unsightly. It would be more difficult to keep the step clean than the gutter, as no large flow would ever reach the step to cleanse it, while flushing and rains would thoroughly cleanse the gutter. Mr. Evans.

The methods outlined by Mr. Tillson are much more advantageous than those described by the author, and this is particularly true of the method of making the step at the stoop or area line. It is a common sight to see a house of the low-basement style where there are four or five steps leading to the basement floor, and from eight to twelve leading up to the first floor. When these houses are located on the higher side of the street, the speaker can see no reason why the sidewalk could not be depressed from 5 to 8 ins., thereby doing away with one or two of the steps down to the basement floor and adding them to the steps to the first floor. This increase in the number of steps to the first floor might be considered an objection, but hardly a serious one, as in the same block the number of steps often varies from three or four to a dozen. It would seem that it could, therefore, be easily adjusted. In business blocks, where there is a longitudinal grade, it is the custom to provide a series of steps in the coping of the area wall

Mr. Evans. which vary from nothing at the upper end to 6 or even 8 ins. at the lower end. It seems that there ought to be no serious objection to using practically the same method on transverse grades, provided the whole street can be improved thereby, and it is manifest that many streets could be improved in this manner.

Numerous petitions for improved pavements are made by residents, yet, so far as the speaker knows, very few of the petitioners are willing to submit to even the slightest inconvenience for the sake of improvement, much less would they be likely to submit to any lowering of the sidewalk, or anything that would touch their pocket-books. The speaker knows of instances where it was even impossible without process of law to get owners to make the slight repairs to their sidewalk made necessary by setting a new curb. It would seem, therefore, that some form of legislation would be necessary in order to provide for these adjustments, in repaving a street.

The speaker recalls an instance where a catch-basin was left stranded on the summit of a pyramid by changing the cross-section of the street at that intersection. This was caused by a difference of some 2 or 3 ft. in the curb levels. The cross-section of the new pavement was the already mentioned "straight line with 1-in. crown." What the old cross-section was, the speaker does not know, but he remembers that from 10 to 20 ins. of material was removed from the old crown, in reconstruction. This left the catch-basin in its exalted position which was determined by the unchangeable law of the sidewalks. Had the authorities been able to carry the improvement to the property line, a step at the area line would have solved the difficulty, improved the street, and kept the drainage from flooding diagonally across the street, deluging the cross-walks and overflowing the shallow gutters on the lower side of the street.

An interesting question is presented by the method of taking care of street drainage mentioned by Mr. Owen, in cases where under-trolleys are operated, for the reason that much, if not all, of the water would flow into the slot and, doubtless, cause much trouble and annoyance in operation. As to whether a city can so construct its streets as to make a storm sewer out of an under-trolley conduit is probably a question for the courts. Granted that it has such a right, it seems hardly fair to citizens and patrons of the road, to say nothing of the company, that they should be subjected to the inconvenience and irritation of blockades incident to "short circuits," "burn-outs," etc., caused by improper city drainage. In New York City, where one, from necessity rather than choice, must live far from business, and depend upon street railways for transportation, the latter should be considered as a necessary adjunct to business rather than a necessary evil.

Mr. Henry. PHILIP W. HENRY, Assoc. M. Am. Soc. C. E. (by letter).—Although the engineering problem to which this paper calls attention is not a difficult one, yet it is one which is met continually by city engineers and is seldom solved satisfactorily. This, however, is not due to its intricacies, from an engineering standpoint, but because the solution is complicated by politics and the influence of property holders. It is an exceedingly difficult task for an engineer to improve a street, where the conditions are similar to those mentioned in the paper, without arousing the determined opposition of those most nearly interested. For the sake of saving a few dollars the average property holder is willing to allow any kind of a grade to be established; and he objects most strenuously to any plan which will cause him to raise or lower his sidewalks or alter in any way his buildings, no matter how old or dilapidated they may be, or no matter how much the improvement may benefit his property eventually. It follows, therefore, that a city engineer is seldom able to carry out his own convictions in establishing a street grade, unless he is willing to make a determined fight, or use all the persuasive powers at his command. In Wabash, it appears that either the engineer's persuasive powers are highly developed, or else the property holders there are unusually enlightened, for the owners of the buildings affected, in several cases, waived all claims for damages occasioned by the change of grade.

It also appears that the author did not attempt too much in improving the grade of the street. The change from the old condition was radical, but at the same time it was no greater than the circumstances warranted. The difference in level between the two sidewalks was met in a somewhat novel manner, by putting in a step on the upper side of the street. It is seldom that this construction is used, and it could doubtless be introduced in other cities to advantage. Its chief disadvantage is that it lessens the width of the roadway, and on a narrow street this would, of course, be objectionable. The ample width of the Wabash roadways allowed this construction, with excellent results. The illustrations on Plate I show what a decided change was made in the appearance of the street by the improvements described in the paper. To one who saw the street before and after the improvement, the change is even more marked than that shown in the photographs.

To return to the details brought out in the paper: It does not appear why the crown was placed in the quarter in Sections 1 and 2, and not in Sections 7 and 8. It is generally objectionable to have all the water drain to one side of the street, as shown in the two latter sections. It would also seem that in these two sections each step could have been made 0.85 ft., the greatest limit, and thus have reduced considerably the slant of the street from one gutter to the other.

Although the property holder has undoubtedly been put to con-

Mr. Henry. siderable expense by the radical change in sidewalk grade, yet it is obvious that any less radical treatment would have given a patch-work effect to the street, which would have been a continual source of annoyance. There are too many streets in prominent cities which are anything but creditable to the engineers who have established the grades. This is generally due to poor engineering talent (if any engineer was employed at all), in laying out the street at the start, for many streets were originally country roads which no one foresaw would one day require paving. By the time the street had arrived at the dignity of a pavement, buildings had been erected at different grades on each side, so that the engineer who had the improvement in charge was at his wits' end to know the best way to deal with the problem. In solving this problem no general rule can be laid down, but each case must be treated by itself, taking into account the many different questions which are involved. Among these questions are width of street, value of real estate and improvements affected, amount and kind of traffic, probable increase in real estate values and probable increase in traffic. These questions are being met every day by municipal engineers all over the country, and are being solved in a better and more satisfactory manner than ever before There is no doubt that the city engineer of to-day is the superior of his predecessor of twenty years ago, and brings greater talent and broader knowledge to bear on the questions which are continually before him. It is to be hoped that the paper will bring out discussions from engineers who, from practical experience, are able to give much interesting and profitable information to the profession.

Mr. Harley. A. F. Harley, M. Am. Soc. C. E. (by letter).—The problem of street grades and cross-sections is, perhaps, one of the first that confronts a city engineer, and is one in which, between his own judgment and the wishes of the various property owners, he is likely to find trouble.

In considering the subject, great difference should be made in the treatment of business and residence streets. Good renting property on business streets should be provided with such sidewalk grades in front of the various stores as to cause the pedestrian to literally roll into the store he may be passing. No such thing as a step should ever be provided near the building line, but if the grade of the sidewalk is not flush with the floor level, the entrance to the store should be "bayed," and a cast-iron foot-plate, connecting the two elevations, should be set in. On business streets it would not be objectionable, if it were found absolutely necessary, to maintain a sidewalk grade even 18 ins. above the gutter. We know that when this is the case, the street, or rather the original surface of the ground, has invariably a hill-side cross-section, and at each end of the block the grade at intersecting streets would make a tread from the sidewalk to the gutter of not over 6 or 8 ins.

Mr. Harley.

It is the writer's observation that store-keepers prefer this high curb, as it requires pedestrians, ladies especially, to promenade the full length of the block and pass other stores, while intending to visit only one in the row. Then, again, the facility for unloading, already mentioned by Mr. Owen, is a point well made.

A municipal engineer should always remember that the great majority of the citizens walking over a street surface do not carry in their heads an engineer's eye, and, so long as the water does not remain on the surface, they could not tell which way it flowed after the rain had ceased, and especially could they not detect a hill-side cross-section, *i. e.*, one in which the summit has been shifted from the center toward the higher side, nor a warped surface that runs out in 100 to 200 ft., spoken of by Mr. North. Differences of elevation on the surface of the street paving proper, as a rule, are not detected, even when the water in the gutters on each side of the street runs in opposite directions for a short distance, as might be the case at the intersection of streets that had grades of say, 4 ins. and 4 ft., respectively.

The writer holds, with Mr. Tillson, that a street should always have some crown, to allow of slight wear to the surface, which is always most in the center, provided there is no electric railroad on the street. But on the other hand, on narrow business streets, say, 30 ft. in width, occupied for more than half their width in the center by two electric street railroad tracks, in the writer's opinion, it is an open question whether it would not be desirable to throw all the water to the center, provided it could be collected and carried off, as it would be the only moving thing that would not meet its death by being run over in crossing the track; while the surface of the street close to the curb line would be more available for vehicular travel; in fact, this gutter space would be gained. In a street railroad, especially one in which the use of the ordinary T-rail is permitted, the paving between the rails is given a crown of, say, 1 in.; thus little streamlets are formed and are compelled to run the entire length of the longitudinal grade on the street, which is bad for both the railroad and the pavement, as it is almost impossible to make water-tight joints between the rails and the adjoining pavement. By the adoption of a central gutter the necessity for making these joints absolutely water-proof is avoided. Were it not for the appearance the writer would like to see the paving between the rails concaved, instead of convexed, and cast-iron gratings having the same concavity placed over catch-basins built in the center of the track at the required intervals.

The writer does not think it would be very much to ask of street railroads using the underground electric system, to make provision in their conduits to carry off the surface water that might be led into them in this manner.

Another point is, that while a very large majority of people, in

Mr. Harley. walking or driving over a street, do not notice the grades on a continuous asphalt surface, as already stated, they readily detect the difference in the height of the curbs on the street over which they are driving; even such a small difference as 4 is to 6 ins. It would, therefore, be advisable on all residence streets to make the height of the exposed curbing the same on both sides of the street, even though the surface of the pavement may have a "hill-side cross-section" or be a "warped surface."

Mr. Andrews. HORACE ANDREWS, M. Am. Soc. C. E. (by letter).—This carefully prepared paper is of interest, and the double curb is an innovation that may be found useful where other means fail. It is much to be regretted that nothing is said either in the paper or in the discussion as to the cost of the improvement. This is certainly a factor of too much importance to be disregarded by a municipal engineer. Concrete curb with gutter attached, similar to that described in the paper, has been laid, under the writer's specifications, for 90 cents per lineal foot. It will be of interest to know whether the double curb cost twice the above amount.

The writer agrees with Messrs. Tillson and Hankinson that the second step would better have been omitted at the curb and placed near the fronts of the buildings.

Two steps might even have been used in such a location. The street once improved is apt to retain its cross-section for many years, while it would appear from the photographic views that many of the buildings will be replaced before very long and then the suggested steps in front of them could be done away with.

The relative widths of sidewalks and carriageway are closely connected with the question of economical improvement. It would seem that the 90-ft. streets would have looked better, would have amply accommodated all traffic and would have been more economically improved with 20-ft. sidewalks and a 50-ft. carriage-way, while the 66-ft. streets, with 30-ft. carriage-ways, would certainly accommodate all vehicle traffic in a place the size of Wabash. It would be much cheaper to construct sidewalks than equal areas of asphalt pavement.

The writer agrees with Mr. North as to the superiority of stone over concrete for curbs. The concrete gutter is peculiarly objectionable on account of the running joint at its outer edge; this is quite apt to wear into a rut, a trouble experienced with all such joints where the wheel traffic is considerable.

It is not apparent that an extra pitch, transversely, was needed for the gutter. The general contour of the street might have been maintained, as usual, to the vertical face of the curb.

It seems to the writer that the author and Mr. Tillson are correct in their objections to an absolutely plane surface for the pavement, although they differ from Mr. North in this respect; but quite as

serious an evil is the level surface resulting from the use of a circular- Mr. Andrews.
arc cross-section, with gutters of equal elevation. Referring to Section No. 12, it is seen that for 7 ft. at the center of the carriage-way the rise is only $\frac{1}{8}$ in. It is practically impossible to avoid depressions along the center of the street where so slight a rise is attempted.

As to the amount of crown to be given a street with gutters of equal elevation, the writer prefers $\frac{1}{72}$ rather than $\frac{1}{100}$ of the width of carriage-way as giving sufficiently flat transverse slopes and as avoiding depressions, while the water is more rapidly brought to the gutters. The ratio $\frac{1}{72}$ is practically that used by the author, the radius of his circular segment then being nine times its chord. The variation between the circle and the parabola is so extremely minute that the cross-section may be termed circular with perfect propriety. The ordinate of the parabola is $y = \frac{x^2}{2r}$, while the ordinate of the circular arc is $y = \frac{x^2}{2r} + \frac{x^4}{8r^3} + \frac{x^6}{16r^5} +$ etc. In the author's example, for the maximum ordinate, $x = 18.67$, $r = 9 \times$ width $= 336.06$, sufficiently near (the author gives $r = 335.30$). For the parabola y will be 0.5186, while the second term, $\frac{x^4}{8r^3}$, increases this only by 0.0004, the third term being too small to be considered.

The writer, a number of years ago, abandoned the circular-arc form of cross-section and has adopted, for some 20 miles of pavement on concrete foundation, a form intermediate between the circular arc and straight lines forming chords of half the arc.

According to Thomas Codrington, in his article on roads,* this form is used in London, and Mr. Hankinson's proportion of 8 to 5, as that which the crown bears to the ordinate at the quarter, also agrees with this cross-section. A useful comparison between this form and the circular arc can be made by considering the rates per cent. of the slopes at different parts of the transverse section. If R represents the ratio of width to crown, 72 : 100, etc., and the carriage-way is transversely divided into eighths; then the rates per cent. in the four divisions lying between the center and the gutter, starting at the center, are: For the circular arc, $\frac{50}{R}$, $\frac{150}{R}$, $\frac{250}{R}$ and $\frac{350}{R}$, and for the improved form mentioned, $\frac{125}{R}$, $\frac{175}{R}$, $\frac{225}{R}$ and $\frac{275}{R}$, respectively. For example, with $R = 72$, the slopes of the circular arc are 0.695%, 2.08%, 3.47% and 4.86%, while the preferred cross-section has slopes of 1.74%, 2.43%, 3.13% and 3.82%, being less flat in the center and flatter at the gutters than the circular arc. By thus considering the rates per cent. of different portions of the cross-section, it is easy to perceive the effect

*Encycl. Britannica.

Mr. Andrews. of unequal heights of gutters if the slope of the line between them is expressed in the rate per cent.

It would be interesting to know from the author how nearly he succeeded, practically, in securing the cross-sections called for in his theoretical investigations. The writer has found it quite impossible in dense, wet clay to form the subgrade to exact shape. A 10-ton roller will make waves and depressions both longitudinal and transverse, yet the concrete and pavement will stand well when the work is complete. In the writer's practice the curbs are first placed, and, by means of a cord stretched between them and measured offsets, the subgrade is approximately formed and is then rolled. The cord is again used and iron rods are driven at the eighths of transverse width; the top of the concrete is marked on these with chalk and the subgrade is then shaved off, generally, or filled up, rarely, until it is exactly the proper distance below the chalk marks. Concrete is then spread and, afterward, the iron rods are pulled up. For a brick pavement fine sand is spread on the concrete, and it is then struck by means of a template resting on rollers running on the curbs. This template is formed with considerable exactness and can be used on widths of carriage-way up to 36 ft. For greater widths a template of half the width of the carriage-way is used, running on one curb and on a timber laid temporarily in the center of the street. These templates have two rollers at each end; the faces of the rollers are slightly wider than the curb thickness. They are swivelled so that one end of the template can be drawn in advance of the other, thus accommodating a varying width of carriage-way of as much as 6 ins. The sand cushion is formed with great nicety by this means; it is then dampened, the bricks are laid and afterward rolled with a 5-ton roller. The pavement is then carefully inspected and imperfect bricks are replaced. The sand cushion is hard and compact, and the contact with the bricks is perfect. The joints are then grouted with Portland cement and sand in equal parts. The writer has laid 13 miles of brick pavement in this manner and has observed no rumbling or distortion such as has been the cause of complaint in some cities. The bricks used have been red shale in all cases.

Mr. Owen's suggestion of a depression in the center of the street is worthy of serious consideration. The writer has often been impressed with the desirability of such a form of cross-section when for several months the gutters are filled with snow and ice, and in times of thaw all drainage is necessarily in the center of the street. Except in the case of alleys where there are no sidewalks the writer has, however, never used a cross-section throwing the water toward the center of the carriage-way.

A low retaining wall $1\frac{1}{2}$ to $2\frac{1}{2}$ ft. high has been used with advantage

in place of a curbstone on the high side of a carriage-way. In Troy, N. Y., there exists such a wall nearly 6 ft. in height. Mr. Andrews.

In the improvement of the streets of the older and of the hilly cities the municipal engineer is often in despair at the difficulties arising from the original and grossly improper formation of the street. The author is to be congratulated on the comparatively easy nature of the problem and on the readiness of the inhabitants, as commendable as unusual, to co-operate with him in the carrying out of his plans.

Robert P. Woods, Jun. Am. Soc. C. E. (by letter).—It appears probable that methods of street construction may vary in different parts of the country and still be suited to their respective localities and attending circumstances. Mr. Woods

The method of shaping unbalanced cross-sections recommended by Mr. Tillson was used by the author on Miami Street (Sections 1 and 2), and would have been used throughout the improvement, but that its continued application would have produced results apparently more unsatisfactory than the double curb.

The five sections as rearranged by Mr. Tillson are not as practical as might appear from a casual inspection of them singly. In his Section No. 2 the east gutter on Miami Street is raised 10 ins., leaving but 2 ins. fall in a block of 280 ft. This, obviously, would not answer.

By raising the west gutter on Wabash Street 1 ft., as shown in his Section No. 5, the grade of the gutter at the alley north of Canal Street would be 1 in. above the sidewalk.

In his Section No. 7 the slope of the south walk on Market Street is lessened to 0.20 ft., which is sufficient for that walk; but the longitudinal grade of the west walk on Wabash Street is not as great as the Market Street walk, and he thereby reduces the cross-slope of the Wabash walk to 0.08 ft. or $\frac{1}{13}$ in. per foot, which is not enough.

The predominating slopes used by Mr. Tillson in the roadway revisions appear to be far too slight for the existing conditions in Wabash, and from this standpoint the author believes they are not acceptable.

The author's Sections Nos. 7 and 8 show the slope of the roadway to be from one gutter to the other, but in connection therewith it should be noted that either of these sections is on a warped or boned-in section, and was exhibited merely to show a partial detail of the manner in which the Market Street grade was merged into the hill-side grade of Wabash Street. At no place in the improvement does water drain transversely of the roadway from one gutter to the opposite; the longitudinal grade of the north gutters on Market Street, at the warped sections cited, each being 3%, the water is carried to them.

It has been suggested that the extra step should have been placed at the building line, or at the area line. To illustrate the local feeling

Mr. Woods. concerning such location, it may be stated that the original grade on the north walk of Market Street from Wabash Street westward 324 ft. was flush with the lower curb; but before this particular walk was constructed the nine abutting property owners signed a petition asking the Common Council to raise the grade by means of the double curb, and at the expense of the abutters. This petition, which had been presented after most of the double-curb work had been constructed, would lead one to presume, that from a local standpoint the curb was not considered "expensive, unsightly and dangerous," as indicated by Mr. Tillson.

The total extra cost of the improvement to the abutting property owners adjacent to the double curb was but 10 cents per foot more than to those along the single curb.

Mr. Evans believes the double curb would be a repository for filth, yet during the nine months that have elapsed since the construction it has not been, nor should it become so, for should the merchants fail to sweep the curb when they sweep the walk, the drainage from the walks, upon the smooth cement surface of the curb, which has a cross-slope of $\frac{1}{4}$ in. per foot, would be ample to keep it in a sanitary condition.

Mr. Owen's preference for a single high step at the curb, when it is necessary, is well founded, where there is to be much transfer of heavy merchandise; but in Wabash the freight is transferred from the wagons to the buildings in the 16-ft. alleys.

It was the peculiar local conditions surrounding the district improved that caused the author to arrange the double curb as used, and the results are decidedly satisfactory.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 854.

COMPARATIVE TESTS OF BITUMINOUS STEAM COALS.

By John W. Hill, M. Am. Soc. C. E.

Presented May 3d, 1899.

WITH DISCUSSION.

During the past few years several papers have appeared in the *Transactions* of the American Society of Mechanical Engineers, and in engineering periodicals, touching the ultimate chemical analysis of coals, especially bituminous and semi-bituminous coals, and upon the determination of the heating power of coals from the proximate and ultimate analyses and by calorimetric methods; and an attempt has been made upon more than one occasion, to deduce from a knowledge of the intrinsic thermal value of a coal the relation between its calorific power and the practical results which can be obtained from it when burned under steam boilers. In one of the most interesting and exhaustive of these papers, contributed by the late Charles E. Emery, M. Am. Soc. C. E., the whole field of experimental information on the comparative efficiencies of coals is briefly and clearly reviewed.*

The papers contributed by Professor R. C. Carpenter, of Cornell University; Mr. George H. Barrus, M. E., of Boston; Professor N. W.

* "Comparative Tests of Steam Boilers with Different Kinds of Coal." *Transactions*, American Society of Mechanical Engineers, Vol. xvii.

Lord, of the Ohio State University, and others, when taken together, seem to indicate a more profound interest in the composition, energy and practical possibilities of steam coals, than has been displayed in all the years that have transpired since Professor Walter R. Johnson made his celebrated investigations for the United States Navy,* and suggest that any experimental data upon this subject should be welcome to those whose interests touch, however indirectly, this important factor of modern civilization.

During the summer of 1897, the author was commissioned by the Honorable Board of Administration, of Cincinnati, to investigate and report on the various bituminous steam coals available in the local market, with a view to obtaining a set of factors by means of which the prices asked by dealers for different kinds and grades of coal could be reduced to a uniform basis for the comparison of bids. The practical application of these factors can best be illustrated by the fact that heretofore it has generally been assumed, upon receiving bids on coal for the City Water Department or any other municipal service, that there was no large difference in the quality of the various coals offered, and that the lowest bid represented the most economical purchase. Each mine owner or agent very naturally has insisted that his coal was equal or superior to that of his competitors, from which the average city official would argue that if one coal is quite as good as another, the lowest bid represented the greatest economy to the corporation. By means of the factors mentioned, the price bid is multipled, and a new set of prices obtained for purposes of comparison only; and that price, which after such reduction is the lowest, represents the most economical coal for steam purposes.

The experimental work embraced:

First.—A practical steam test of each kind and grade of coal (of which there were over 50), under uniform conditions, for a period of 16 hours.

Second.—A proximate chemical analysis of each kind of coal.

Third.—A calorimetric determination of the heating power of each kind of coal.

The investigations, with some interruptions, occupied the interval from July 27th, 1897, to April 28th, 1898, the evaporation tests being conducted at the Hunt Street Pumping Station of the City Water-Works, and the laboratory work in the author's office in the Glenn

* Washington Navy Yard, 1842-43.

Building. In the physical tests, from 18 000 to 25 000 lbs. of coal were burned during a continuous trial of 16 hrs., and from 168 000 to 178 000 lbs. of water were supplied to the boilers.

The duration of the physical tests, 16 hrs., was fixed upon in order to embrace two full watches or tricks of the engineers, firemen and coal wheelers employed at the station. All the different kinds of coals were fired under the same boilers by the same firemen, one fireman firing during the 8-hour morning watch, and the other during the 8-hour evening watch, the relative position of the men being changed at the end of each series of eight tests.

The coals were burned in a pair of Galloway fire-box boilers, Fig. 1, built by the Edge Moor Iron Company, of Wilmington, Del. These boilers had been in service at the station for ten years prior to their use for the coal tests, but were in excellent order, and during the period of these tests were used for no other purpose.

The physical tests contemplated the same amount of work per hour for each kind and grade of coal, the amount of coal burned to perform this work to be varied according to its quality. With some of the inferior coals, especially with "Slack," this could not always be accomplished, and the rate of steam production was therefore lowered slightly to admit of burning such coals under fair conditions. The steam was used in operating the pumping engines in the station, which were under the author's control, in order that the consumption of steam should keep pace with the rate of production adopted for the tests. The whole work of the station during the interval of the physical tests was nicely adapted to the requirements and conditions of the coals submitted to investigation.

To avoid as much as possible the furnishing of selected samples of coal for test purposes, the city purchased the coal under the formal requisitions of the Water Department, and the dealers were in no instance permitted to be present during, or have knowledge of the results of, the trials. It was hoped thus to arrive at the fair commercial value of the respective coals, and to eliminate from the economic results the influence of samples selected for the occasion.

The two firemen who stood all watches during the tests were detailed by the Superintendent of Water-Works for this special duty. All assistance specially required for the tests was furnished from the author's office.

The two Galloway boilers were each of the following dimensions:

Shell, 7 ft. diameter, 28 ft. long.

Fire-boxes (2), 2 ft. 10 ins. inside diameter, 8 ft. long.

Grates (2), 2 ft. 9 ins. wide, 6 ft. 7 ins. long.

Combustion flue, 2 ft. 11¾ ins. vertical diameter, and 6 ft. 2 ins. horizontal diameter, 20 ft. long.

Galloway circulating tubes (33), 6 ins. diameter small end, 10½ ins. diameter large end, 2 ft. 11¾ ins. long.

The boilers were set so as to expose about two-thirds of the shell as heating surface. The heating surface of one boiler is divided as follows:

Heating surface, fire-boxes (2)	83.982	sq. ft.
" " combustion flue	296.935	"
Heating surface, Galloway tubes (33)	212.338	sq. ft.
" Rear head	20.916	"
" Shell	381.170	"
" Total	995.341	"
" Two boilers	1 990.68	"
Cross-section, Two fire boxes	12.608	"
" Combustion flue	11.444	"
" Smoke tunnel	15.155	"
" Sheet-iron chimney	12.566	"
Grate surface (2)	36.206	"

Ratio, heating to grate surface.....27.50 to 1

Ratio, grate surface to flue vent..... 3.16 to 1

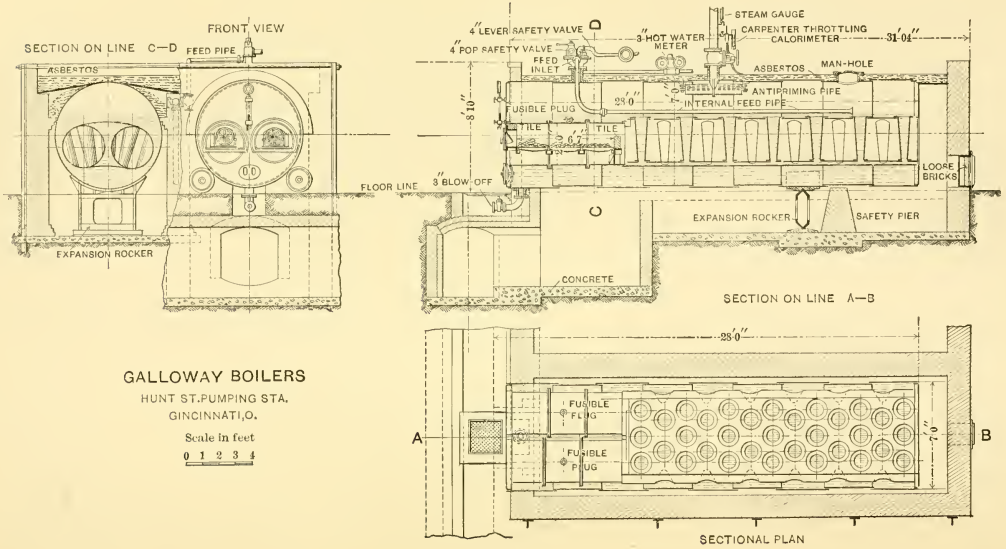
Water surface, one boiler (mean level) 6 ft. 1½ ins. wide, 28 ft. long.

Chimney (iron), 48 ins. diameter, 70 ft. high.

The length of the smoke traverse from the front end of the grates to the top of the chimney was as follows:

Boiler No. 1	162.37 ft.
Boiler No. 2	153.21 "
Separation of boiler centers	9 ft. 2 in.

On October 11th, during Test No. 28, and on November 18th, during Test No. 35, tests were made to ascertain the time required by the hot gases to flow from the front end of the fire-boxes to the



GALLOWAY BOILERS

HUNT ST. PUMPING STA.

CINCINNATI, O.

Scale in feet

0 1 2 3 4

FIG. 1.

top of the chimney. This was accomplished by allowing the coal on the grates to burn to incandescence, and then throwing into the fire-box a pint of liquid asphaltum and quickly closing the fire-door. From these tests, the mean velocities of the gases through the fire-boxes, flues and chimney were as follows:

Boiler No. 1	10.825 ft. per second.
Boiler No. 2	11.785 "

The Galloway boilers were the only ones in the water department which were not supplied with some kind of a fuel-saving device or some other appliance which might interfere with a uniform treatment of all coals, and also be of sufficient capacity to admit of using the coals in reasonably large quantities. These boilers, without urging, made dry steam at the rate of 10 625 lbs. per hour from feed water at a temperature of 190° Fahr., and with a steam pressure of 102 lbs.; equivalent to 326.90 Commercial H. P. by the Centennial Standard of boiler power rating.

The coal was weighed from the sample lot in iron barrows carrying from 1 250 to 1 600 lbs., depending upon the kind of coal, and was fired directly from the barrows to the boilers. The time each barrow was fired from was noted, and the unburned coal in the barrow at the end of the test was weighed back and deducted from the aggregate charge.

Provision was made, upon receiving a sample shipment of coal at the pumping station, to store it so that rain or snow should not wet it, and each sample was kept separate from the coal used for the routine work of the station. The tare of the iron barrows was taken each time one was run out of the boiler house empty for a new load of coal, and the net weights so obtained were the weights of coal actually fired from each barrow.

Previous to the beginning of each test, the boilers, with clean grates, were fired from the sample coal, the steam pressure was brought to 102 lbs. by gauge, the temperature of the feed water was raised to 190° Fahr. (with few exceptions), and the pumping engine was started at normal speed. This procedure was always observed, and occupied usually about two hours.

When the charges of coal on the grates, just before starting a measured test, were no longer able to maintain the steam pressure

under normal conditions of engine speed, temperature, and rate of supply of feed water, the signal for starting was given, and for 16 hrs. thereafter all notes necessary to make a physical test of the coals were taken half-hourly.

At the end of each test the last charges of coal on the grates were continued, as at the beginning, until the steam pressure, with the normal speed of the pumping engine, could no longer be maintained, when the test was stopped.

The grates were cleaned, usually, at regular intervals, and all ash, clinker and unburnt coal was weighed back dry. The firemen were constantly under espionage during the tests, and were required to be alert and attentive, without exceeding the care which at all times should prevail in firing coal under steam boilers.

The water was supplied to the boilers through a pair of 3-in. hot-water, Worthington meters, made for the purpose, which were rated before and after each series of eight tests, and occasionally on shorter test intervals. The meters were rated under the same conditions of temperature of water and rate of delivery as during the coal tests, the water discharged being weighed in a tight tank on a platform scale, and reduced to volume corresponding to the temperature of the water passing through the meter. The meter tests in each series were sufficiently numerous to avoid error in determining the value of the factor required to reduce the meter reading to the true amount of water delivered to the boilers for the previous series of coal tests.

In anticipation of some criticism on the use of water meters for the measurement of water to the boilers, the following explanation of the conditions which brought about their use in preference to tanks and scales is offered:

The pumping station where these coal tests were conducted is in daily service, although usually for a shorter length of time each day than the interval of preparation for and test of each sample of coal, and upon the odd days when no tests of coal were in progress, the machinery was operated in the usual service, and at the completion of each coal test, the house was left in such condition that the pumps could be operated on the following morning, taking steam, however, from one or two of three other sets of boilers in the house, the Galloway boilers being reserved exclusively for the coal tests.

The feed pumps and pipes and feed-water heater were common to

all boilers, but during the test of a sample of coal were devoted exclusively to the test boilers, and at all other times to such other boilers as may have been in service. This arrangement prevented the usual breaking of connections in the general system of feed-water pipes to the ordinary service boilers, and compelled a measurement of the water to the test boilers at a point in the line of feed-water pipes which would effectually avoid any gain or loss of water before it was delivered to the test boilers; and as at such point (on top of the boilers) the water had passed through the feed-water heater and been brought to near the boiling point, meters were regarded as the only convenient means for this purpose.

The feed pipes from the meters to the check valves of the Galloway boilers were exposed to view, and no loss at joints could have occurred without its being promptly noticed; and no connections were made with these pipes through which either gains or losses from other boilers might have occurred. Thus all water which passed through the meters was delivered without gain or loss into the test boilers.

Recognizing the difficulty of a satisfactory measurement by meter of the water delivered to the boilers, careful tests of the meters were made previous to the coal tests, and at frequent intervals during the first few tests, until it was discovered that the wear of the meters followed a nearly uniformly varying curve; and in determining the amount of water to be credited to any test, it was assumed that the wear of the meters was uniformly variable between any two successive tests of the meters, and the gross error was properly distributed between the several preceding tests.

In testing the meter for error of registry, 25 cu. ft. (by meter) was taken for each draft in the tank, and usually five or six drafts were made to correct for slight variation in rates of flow. The error was obtained by comparison of the actual weight of water drawn with the weight at the observed temperature of the water registered by the meter.

Previous to each test and at frequent intervals the screws in the top of the meter case were loosened to vent any air or vapor which might have formed in the meter, but, excepting upon starting, there was seldom any air to vent, while the temperature of the water (190° Fahr.), and the pressure under which the meters were worked (105 lbs.) precluded the formation of vapor from the hot water.

The coal tests were for comparative rather than absolute effects, and so long as the method of water measurement was alike for all samples, and not subject to vagaries or phenomena, it was considered sufficient for the purpose. Meter readings were taken and compared half-hourly during the whole test of each sample of coal.

The water levels in each boiler were read from scales attached to the glass tubes and graduated to eighths of an inch. Any discrepancy between the first and last water levels was corrected by adding or subtracting the proper volume of water reduced to weight due to the temperature of the steam and water in the boilers at the end of test. By careful management of the boiler feeder it was often possible to obtain exactly the same water level at the end of the test as at starting, and during the tests to hold the water levels quite close to the starting point.

The readings, as taken for the two boilers, indicated the depth of water above the brass nuts at the lower ends of the glass gauges. These nuts were 1.875 ins. below the crowns of the fire-boxes, and this amount subtracted from the readings on the scales gives the depth of water over the fire-boxes during the respective trials. In the tables which follow, the water levels are stated as the mean depth in inches over the fire-boxes.

The steam gauges and thermometers were of good make, and were compared from time to time with standard test instruments, and all readings were corrected for errors of registry. The quality of the steam was determined by a pair of "Carpenter" throttling steam calorimeters, obtained from the Schäffer and Budenberg Company, New York. One of each of these was set in the steam outlet at the midlength of each boiler, the deductions were averaged from each instrument, and the mean result used in computing the water entrained and the net steam for each test.

Calorimetric observations of the steam were taken every half hour during each test, although it was early discovered that the condition of the steam was a function of the boiler and not of the coal, the quality being quite the same for good and bad coals, and such variations as occurred from time to time during any test, and from coal to coal during the several months of test, were attributed to variations in the rate of steam flow to the pumping engines, and to the opening of the fire-doors at the time of taking calorimeter readings. The calorimeters

and steam connections were heavily clothed with non-conducting materials to prevent loss of heat to the atmosphere, or gain of heat from the steam pipe to the condensing bulb of the instrument.

During these tests over 3 600 observations were made with the steam calorimeters, and these were reduced and carefully studied for influence of the pumping engines on the quality of the steam.

Efficiency of the Steam.

With reference to the quality of steam, this should depend upon the type and setting of boilers, the rate of evaporation per unit of heating surface, and the manner in which the steam is taken from the boilers. Super-heating surface will be required to produce dry or super-heated steam, and a lack of super-heating surface in the design or setting of boilers means steam of thermal value lower than saturation. The quality of steam may be slightly affected by the fuel and firing. Thus a low-grade fuel, or careless firing, with frequent opening of the fire-doors and the rush of cold air into the fire-boxes or furnaces, may lower the quality of the steam and increase the amount of water entrained. But with coals not varying greatly in value, and with uniform care in firing, the quality of the steam should be a function of the boilers, the rate at which they are worked and the manner of taking the steam from the boilers.

If the steam is delivered at a steady rate into the atmosphere, or to an engine, where there is no sensible variation in the flow for any given rate of evaporation, the quality should be a maximum. When the delivery of steam is irregular and subject to large and rapid variations of flow, the quality should be a minimum.

At the time these tests were projected, it was thought that the quality of the steam would be quite uniform for all tests, varying inversely with the rate of evaporation, and rising and falling slightly with the quality of the coals; but no consideration had been given to the influence of the engines on the quality of the steam.

The pumping station contains two pumping engines, each of the same nominal capacity, viz., 6 000 000 galls. per 24 hours, delivered against a net head of 310 ft. One of these is a plain compound condensing Worthington engine, and the other (the Niles engine), a pair of non-condensing western river steamboat style of engines, coupled, with cranks set at quarters, and steamboat poppet valves operated by

rocking toes and lifters. The operation of the steam valves in the Worthington engine is smooth, and without sensible shock to the column of steam in the steam pipe moving from the boilers to the engine, while the steam valves of the other engine are abruptly opened and closed, producing, while the valves are open, a high velocity in the steam pipe, and bringing the column of steam quickly to a state of rest when the poppet valves are closed. Both the opening and closing movements of the steam valves of the Niles engine are rapidly effected by quick-acting cams, wholly unlike the smooth easy motion of the slide valves in the duplex Worthington engine.

When the steam was taken from the boilers by the Worthington engine, the quality was high; and when taken by the Niles engine, the efficiency of the steam dropped from 1 to 1½ per cent. The fact had been observed for some time before the cause was suspected; but the uniformly higher quality of steam when the Worthington engine was in service demonstrated that the engine itself had an influence on the quality of the steam, and a probable influence on the economical effect of the coal.

Of the fifty-four tests, twenty-four were made with the Niles engine, three partly with the Worthington engine and partly with the Niles engine, and one trial was made partly with the Worthington engine, and partly by blowing the steam through the feed-water heater into the atmosphere. The other twenty-six tests were made with the Worthington engine.

With the exception of Test No. 50 (which was purposely made partly with each engine), changes from one engine to the other, or in the mode of disposing of the steam, were caused by accidents, which compelled the stopping of the damaged engine and the starting of the other; or, as in one instance, the wasting of the steam into the air for a few hours.

Test No. 1 was made for 11 hrs. 56 mins. with the steam going to the Worthington engine, and for 4 hrs. 14 mins. with the steam going through the feed-water heater to the atmosphere.

Test No. 2 was made for 5 hrs. 8 mins. with the steam going to the Worthington engine, and for 11 hrs. 3 mins. with the steam going to the Niles engine.

Tests Nos. 3 to 19, inclusive, were made with the Niles engine.

Test No. 20 was made for 7 hrs. 13 mins. with the Niles engine, and for 8 hrs. 47 mins. with the Worthington engine.

Tests Nos. 21 to 41, inclusive, were made with the Worthington engine.

Tests Nos. 42 to 49, inclusive, were made with the Niles engine.

Test No. 50 was made for 10 hrs. with the Worthington engine, and for 5 hrs. 41 mins. with the Niles engine.

Tests Nos. 51 to 55, inclusive, were made with the Worthington engine.

Tests Nos. 2, 20 and 50, having been made partly with the Niles engine and partly with the Worthington engine, an opportunity was offered to inquire into the influence (if any) of the engines on the efficiency of the coal.

TABLE No. 1.—Comparison of Effects with Worthington and Niles Engines, Respectively, in Service.

Number of test.	Engine in service.	Pounds of coal per cubic foot of water supplied to boilers.
2	Worthington.	6.549
2	Niles.	7.056
20	Worthington.	7.132
20	Niles.	7.132
50	Worthington.	6.682
50	Niles.	6.696

During the early part of Tests Nos. 2 and 50, the Worthington engine was in service, and during the latter part the Niles engine. During the early part of Test No. 20 the Niles engine was in service, and during the latter part the Worthington engine. Considering these tests, there is a distinct, though rough, indication that the engine through which the steam was worked had an influence on the economy of the several fuels. Comparisons of steam, water and coal supplied to the boilers during intervals of the tests when both engines were (at times) in service, show, in all instances but one, that the coal per unit of water was greatest when the Niles engine was running, while in the single exception the coal per unit of water was almost precisely the same for each engine.

The notes from the calorimeters uniformly show a lower efficiency when the steam was going to the Niles engine, and for a while it was thought that the loss in steam efficiency was probably compensated by

a corresponding increase in the delivery of water to the boilers; but a careful review and comparison of the coal and water supplied to the boilers when both engines—at different times—were in service, show that not only was the quality of the steam lower, but the coal required

TABLE No. 2.—Efficiencies of the Steam.

1 No. of test.	2 Coal under test.	3 Worthington engine and to atmosphere.	4 Niles engine.	5 Worthington engine.	6 Worthington and Niles engines.
1....	New River, Mine Run	0.9898			
2....	Kanawha, Steam Lump				0.9873
3....	Kanawha, Gas Lump		0.9874		
5....	Pittsburg, Catsburg, Lump		0.9880		
6....	" " Barge Run		0.9836		
7....	" " Mine Run		0.9825		
8....	" " Nut and Slack		0.9788		
9....	" Woods, Lump		0.9824		
10....	" " Barge Run		0.9794		(Mean of
11....	" " Mine Run		0.9821		Tests Nos.
12....	" " Nut and Slack		0.9818		3 to 19,
13....	" Brown, Lump		0.9845		0.9821.)
14....	" " Mine Run		0.9854		
15....	" " Barge Run		0.9803		
16....	" " Nut and Slack		0.9828		
17....	" Cin'ti, Mine Run		0.9797		
18....	" " Nut and Slack		0.9782		
19....	Campbell's Creek, Nut and Slack		0.9770		
20....	Loup Creek, Mine Run				0.9851
21....	" " Nut and Slack			0.9901	
22....	Pocahontas, Mine Run			0.9902	
23....	Winifrede, Lump			0.9903	
24....	" Mine Run			0.9906	
25....	" Nut and Slack			0.9902	
26....	Pocahontas, Lump			0.9900	
27....	Loup Creek, Lump			0.9929	
28....	Thacker, Mine Run			0.9956	
29....	Campbell's Creek, Mine Run			0.9956	(Mean of
30....	Thacker, Nut and Slack			0.9949	Tests Nos.
31....	New River, Mine Run			0.9935	21 to 41,
32....	Acme, Mine Run			0.9930	0.9930.)
33....	" Nut and Slack			0.9913	
34....	Keystone, Mine Run			0.9939	
35....	New River, Lump			0.9933	
36....	Monongah, Mine Run			0.9935	
37....	Thacker, Lump			0.9937	
38....	Belmont Splint, Mine Run			0.9949	
39....	" " Nut and Slack			0.9945	
40....	Jellico, Ky., Nut and Slack			0.9913	
41....	Luhrig, Washed Egg			0.9949	
42....	Logan Cons., Thacker, Mine Run		0.9851		
43....	Luhrig, Washed Slack		0.9835		
44....	Wellston Shaft, Mine Run		0.9805		(Mean of
45....	" " Nut and Slack		0.9769		Tests Nos.
46....	Maritime, Thacker, Mine Run		0.9767		42 to 49,
47....	Jellico, Ky., Mine Run		0.9793		0.9797.)
48....	Jellico, Tenn., Nut and Slack		0.9784		
49....	New River, Lump		0.9771		
50....	Thacker, Mine Run				0.9834
51....	Eagle, Mine Run				
52....	Cedar Grove, Mine Run,				
53....	Mt. Carbon, Mine Run,			0.9930	
54....	Chesapeake, Mine Run.				
55....	Coalburg, Mine Run				

per unit of water supplied to the boilers was higher with the Niles engine.

Table No. 2 shows the efficiencies of steam for Tests Nos. 1 to 50, inclusive. In Column 3 is given the mean efficiency of the steam when it was consumed for the major portion of the test by the Worthington engine, and blown through the feed-water heater into the air for the latter portion of the test. Column 4 shows the efficiencies of the steam when the Niles engine alone was working. Column 5 contains the same data when the Worthington engine alone was working; and Column 6 the efficiencies when the steam was taken for a portion of the test by the Niles engine, and for a portion of the test by the Worthington engine.

For Tests Nos. 51 to 55, inclusive (Worthington engine), the calorimeters were read regularly as in all previous tests, but, the notes simply confirming the results obtained from Tests Nos. 21 to 41, inclusive, the mean efficiency (0.993) for these former tests was used in determining the weight of water actually converted into steam for the last five coals.

It was unfortunate that all coals could not have been tested with the same engine taking the steam, but accidents, first to one engine and then to the other, made this an impossibility, and recourse must be had to such corrections for the engines as the data will warrant.

It will be understood that, throughout the tests mentioned, when first one engine and then the other was used to work off the steam made by the test boilers, the same coal was burned, and when comparisons are made of the influence of the engines on the coal required per unit of water, coal always of the same commercial quality is under consideration.

Considering the New River coals, which were tested first (Test No. 1), by working the steam for nearly 12 hours through the Worthington engine, and then for over 4 hours through the feed-water heater to the atmosphere; next (Tests Nos. 31 and 35), when the Worthington engine used the steam for the whole of each test; and finally (Test No. 49), when the Niles engine used the steam for the entire trial, it appears that the efficiency of the steam varied perceptibly under these different conditions of use, and that while the boilers, firemen and fuel were not changed, the change in the manner of using or wasting the steam had an influence on its quality.

TABLE No. 3.—Efficiency of Steam with New River Coals.

Test No.	Kind of coal.	Steam pressure. In pounds.	Disposition of steam.	Efficiency, saturated steam being taken as unity.
1....	New River, Mine Run..	100.5	To Worthington engine, 11 hrs. 56 mins.; to feed-water heater, 4 hrs. 14 mins.....	0.9898
31....	New River, Mine Run..	101.4	To Worthington engine.....	0.9935
35...	New River, Lump.....	101.3	To Worthington engine	0.9933
49....	New River, Lump.....	100.9	To Niles engine.....	0.9771

By this comparison it appears that the efficiency of the steam was 1.63% higher when the Worthington engine was used than when the Niles engine was used, and to avoid prejudice to any coal investigated during these tests, the author concluded that a careful study of all the conditions and data warranted the assumption that the extreme variations in the efficiency of the steam were not due either to the kind or grade of coal burned, but to the manner of using the steam from the boilers, and that for purposes of comparison the coals should be rated upon a uniform efficiency of the steam.

Condition of the Boilers.

The tests were made with the boilers in ordinary condition, and at the end of each series of three or four tests, the boilers were opened and all loose material washed out. The feed water was passed through an Excelsior filter and feed-water heater, which intercepted the mud and heavier suspended matter before it reached the boilers. At each periodical washing a small amount of loose scale and fine silt was removed from the circulating tubes and the bottom of the boilers. After Test No. 49 the boilers were opened and the small amount of scale found was carefully chipped and scraped off to the surface of the metal. Test No. 50 was then made, using a coal the value of which had been previously established during Test No. 28 with the boilers in ordinary condition.

Assuming that Test No. 28 represented the boilers in the condition for all coals tested, while Test No. 50 represented the boilers in the best possible condition, Table No. 4 shows the comparison of effects for these two tests.

TABLE No. 4.—Comparison of Steam per Pound of Coal and per Pound of Combustible for Tests Nos. 28 and 50 (in both using Thacker, Mine-Run Coal).

Number of Test.	28	50
Duration of test.	16 hrs. 00 mins.	15 hrs. 41 mins.
Steam per pound of coal from and at 212° Fahr, Pounds	9.392	9.735
Steam per pound of combustible from and at 212° Fahr. Pounds	10.132	10.357
Percentage of combustible	92.707	93.993
Increased percentage of combustible (second test of coal)	.....	1.387
Apparent gain in economy by coal. Percentage	.....	3.650
Net gain in economy by coal. Percentage	.....	2.263
Gain in economy by combustible. Percentage	.....	2.220

From this comparison it appears that the difference between the performance of a given coal with the boilers very carefully cleaned, and with the boilers in ordinary working condition, was about $2\frac{1}{4}\%$, an amount by which the economic and capacity results obtained with each coal may be increased to show the possibilities of these several coals with perfectly clean heating surfaces and ordinary firing.

Sources of Coal Samples.

Fifty-four samples of coal were tested, as follows:

Fourteen samples of Pennsylvania (Pittsburg) coals.

Thirty-three samples of West Virginia coals.

Four samples of Ohio (Hocking Valley and Jackson) coals.

Two samples of Kentucky coals.

One sample of Tennessee coal.

The Pennsylvania coals were as follows:

Four samples from the Brown Mines, Nos. 10 and 11, Second Pool, Monongahela River.

Four samples from the Catsburg Mine, No. 47, Third Pool, Monongahela River.

Two samples from the Cincinnati Mines, Third Pool, Monongahela River.

Four samples from the Woods Mine, No. 75, Fourth Pool, Monongahela River.

The West Virginia coals were as follows:

Four samples from New River, Mines Nos. 71 to 80, inclusive, Fayette Co.

Three samples from Loup Creek, Mines Nos. 81 to 87, inclusive, Fayette Co.

Two samples from Pocahontas, Mines Nos. 96 to 140, inclusive, McDowell and Mercer Counties.

One sample, Kanawha steam coal from Mine No. 47, Kanawha Co.

One sample, Kanawha gas coal from Mine No. 102, Kanawha Co.

Two samples, Acme coal from Mine No. 24, Kanawha Co.*

One sample, Keystone coal from Mine No. 23, Kanawha Co.*

Three samples, Winifrede coal from Mines Nos. 14, 17, 18 and 19, Kanawha Co.

Four samples, Thacker coal, Mingo Co.

One sample, Logan Consolidated Thacker coal, Mingo Co.

One sample, Maritime Thacker coal, Mingo Co.

Two samples, Campbell's Creek coal from Mines Nos. 10 to 12 inclusive, Kanawha Co.

One sample, Monongah coal from Mines Nos. 2 to 5, inclusive, Marion Co.

One sample, Eagle coal from Mine No. 5, Kanawha Co.

One sample, Cedar Grove coal from Mine No. 27, Kanawha Co.

One sample, Mt. Carbon coal from Mines Nos. 48 and 49, Fayette Co.

One sample, Chesapeake coal from Mine No. 36, Kanawha Co.

One sample, Coalburg coal from Mine No. 21, Kanawha Co.

Two samples, Belmont Splint coal from Mine No. 32, Kanawha Co.

The Ohio coals were as follows:

Two samples, Luhrig coal from Mine No. 1 (Daleton), Athens Co.

Two samples, Wellston Shaft coal from Mine No. 3, Jackson Co.

The Kentucky coals were as follows:

Two samples from the Jellico Mines in Whitly Co.

The single sample of Tennessee coal was from the Jellico Mines, in Campbell Co.

The sketch map (Fig. 2) indicates the geographical location of the various coal fields from which the test samples of coal were obtained.

* These coals represent two separate and distinct veins.

In each instance the samples of coal were ordered from the agents or mine owners in quantity sufficient for the preliminary firing of the boilers and for the 16-hour test which followed, and each agent or mine owner was required to furnish with the sample of coal a sworn statement of the source and market grade of coal supplied. While this may not in all cases have prevented the substitution of coal from one mine for that of another, it put the coal men on their honor to supply the kind and grade of coal ordered by the city for test purposes.

The importance of these tests to Cincinnati will appear when it is known that the City Government alone is a purchaser of over $80 000 worth of coal annually, and the value of the entire consumption of coal for all purposes in the city at the present time cannot be less than $2 500 000 per annum.

The most extensive series of physical tests of coal under uniform conditions, previous to these, of which the author is aware, were made for the United States Navy by Professor W. R. Johnson, in 1842 and 1843. These comprised physical and chemical examinations, and calorific determinations by the Berthier method, of forty-four different samples of coal. In view of the very large quantities of coal burned and water evaporated during each of the author's trials, it is amusing to read from Professor Johnson's report, his description of the magnitude of these earlier (physical) trials for the Navy:

"The total weight of coal consumed in the trials of evaporative power has been nearly 62.5 tons, and the weight used on an average 978 lbs. per trial. This statement may be sufficient to indicate that the experiments have been made on a scale unobjectionable on the score of magnitude."

Presuming that few members of the Society have any knowledge of these earlier coal tests by Professor Johnson, it will be interesting to such to note that the experiments were conducted with a single return-flue boiler, 3 ft. 6 ins. in diameter and 30 ft. long, with two return flues the full length of the boiler, each 10 ins. in diameter. The air to support combustion was heated before it passed under the grate, in flues built in the side walls of the boiler setting, and the gases of combustion passed from the grate backward under the shell of the boiler, thence through the flues to the front end of the boiler, thence to the rear end of the boiler through a flue in the brickwork on the left side above the fire line, and thence forward through a similar flue in the right side of the setting, to the base of the brick chimney, set

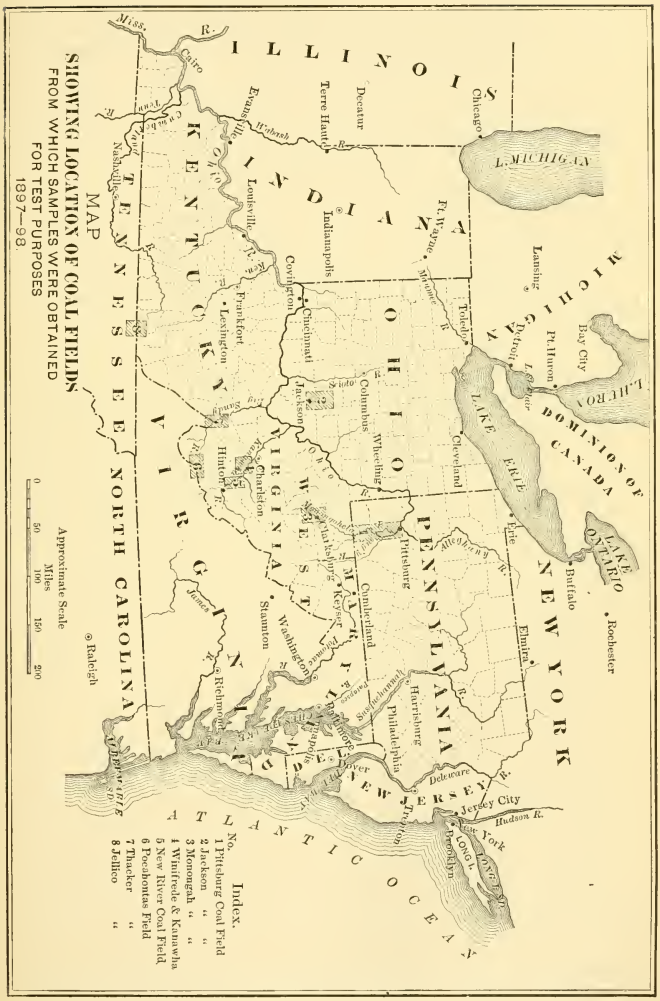

MAP
SHOWING LOCATION OF COAL FIELDS
FROM WHICH SAMPLES WERE OBTAINED
FOR TEST PURPOSES
1897—98.
Index.
No.
1 Pittsburg Coal Field
2 Jackson " "
3 Monongah " "
4 Winifrede & Kanawha
5 New River Coal Field
6 Pocahontas Field
7 Thacker "
8 Jellico "
Approximate Scale
Miles
0 50 100 150 200
ILLINOIS
INDIANA
OHIO
KENTUCKY
TENNESSEE
VIRGINIA
WEST VIRGINIA
NORTH CAROLINA
PENNSYLVANIA
NEW YORK
NEW JERSEY
MICHIGAN
DOMINION OF CANADA
L. MICHIGAN
L. HURON
LAKE ERIE
LAKE ONTARIO
ATLANTIC OCEAN
Chicago
Decatur
Terre Haute
Evansville
Louisville
Indianapolis
Ft. Wayne
Toledo
Lansing
Bay City
Pt. Huron
Detroit
Covington
Cincinnati
Frankfort
Lexington
Columbus
Wheeling
Cleveland
Erie
Buffalo
Rochester
Elmira
Pittsburg
Cumberland
Keyser
Staunton
Washington
Charleston
Hinton
Richmond
Raleigh
Harrisburg
Philadelphia
Baltimore
Trenton
Dover
Jersey City
New York
Brooklyn
Cairo
Nashville
Miss. R.
Wabash R.
Ohio R.
Kan. R.
Cumberland R.
Tenn. R.
Sandy R.
Kanawha R.
Monongahela R.
Allegheny R.
Susquehanna R.
Delaware R.
Hudson R.
James R.
Potomac R.
Rappahannock R.
Scioto R.
Maumee R.
Lake Superior
Long I.
Delaware Bay

FIG. 2.

at the front end and to the right of the boiler. The chimney was of brick, 41 ft. high from ground level, with a flue 18 ins. square, and a sheet-iron extension 22 ft. high and 23 ins. in diameter This boiler was worked at an average of about 6.25 lbs. per square inch above atmospheric pressure—the pressure never being less than 5.81lbs.—and upon two occasions only, during this long series of trials, the pressure was raised to 15.65 and 19.16 lbs., respectively. All steam was blown through two direct-weighted safety valves, into the atmosphere or into the chimney.

Considering the low steam pressures used, and the moderate rate of evaporation per unit of heating surface, the capacity of this old-style boiler during the coal tests was about 30 Commercial H. P.

Physical Tests of Coals.

In the following tables giving the *résumé* of each test, the coals have been grouped by States, and the several grades of each kind of coal tested are brought together for comparison of economic results. The favorite grades of steam coals are Mine Run and Nut and Slack, with an extensive use of Slack coal for mechanical stokers. The prices of the coals grade downward from Screened Lump to Slack, and it is constantly a problem with buyers of large quantities of coal to determine which grade at given prices will represent the best economy. Each grade of coal has its advocates among buyers. In spite of the millions of tons of graded coals which have been burned on western river steamboats, it is still a matter of controversy among river men whether Mine Run or Screened Lump coal, at the prices usually charged, is the cheaper coal for steam purposes.

It is customary when burning low-grade bituminous coals to wet the coal slightly before it is charged into the furnace. With many of such coals, this preliminary wetting causes the smaller particles to adhere together until sufficiently heated to make a coke, and prevents the usual sifting of these particles through the grates into the ash-pits with corresponding loss of useful effect; and if the coal be not too dirty, it is often possible by this method to obtain quite as economical results with Slack as with Nut and Slack coal. Owing to the difficulty of determining just how much water each particular kind of coal might take without positive injury to its efficiency, during these tests, the low-grade coals were always fired without pre-

liminary wetting, and from the dealers' point of view, the coal should be valued without regard to subsequent treatment by the buyer. The coals, therefore, in all cases were burned in the same condition as supplied to the station by the dealers or mine owners.

Previous to each trial, 100 lbs. of coal was taken from the sample, and put into a wooden box perforated on the top, bottom and sides, to allow free circulation of air around and through the coal, and placed on one of the steam manheads of the boilers to dry the coal and obtain a factor for estimation of the superficial moisture it contained. At the end of the test this box and sample was returned to the scale, and any loss of weight was taken as the excess of moisture in the coal. The losses estimated in this manner ranged from 0 to 2%, and have been allowed for in the item "Commercially dry coal burned."

The results given in the tables are in no case supposed to indicate the best possibilities with these coals. The firemen were perhaps the best regularly employed by the City Water Department, and carefully and faithfully performed their duties during these tests, but no attempt at expert firing was permitted, the men being directed only to give uniformly good service throughout each and every test. No extra help was furnished for handling the coal, and as a whole, the results are no better than good firemen should obtain in steady service from day to day, with the same coals. The author regards the results obtained by this method of handling the coal as more satisfactory to the buyers and users of coal than those which might have been obtained by expert firemen with assistants to open and close the fire doors, and an amount of labor which men could not maintain day after day.

Referring to Tables Nos. 5 to 11, inclusive, when the "duration of test" (1) as in six instances, was a few minutes more or less than 16 hours, this was occasioned by the steam pressure or water levels in the boilers being too high or too low, or by the condition of the charges of coal on the grates, which compelled an earlier or later stopping of the respective tests. In each test an attempt was made to restore the starting conditions of steam pressures, water levels, fires on grates and speed of pumping engine at the expiration of 16 hours of test, and with the exceptions noted, this was always accomplished. The steam gauges (2), and thermometers (6), (7), (9), were compared for each test with

standard instruments. The temperature of the waste gases in the chimney (4) was taken with a Brown hot-blast pyrometer, which was adjusted before each test by a standard mercury thermometer.

"Commercially dry coal" (10) means the actual weight of coal fired to the boilers, reduced by the amount of moisture lost by a sample of 100 lbs. carried for 16 hours in a perforated box on top of the boilers. This does not represent the moisture found by chemical analysis of the coal samples, but the wetness due to exposure to the weather in barges and railway cars. During the hot, dry weather of August and September, 1897, but few of the coals showed any loss when placed in the box over the boilers. The conditions with reference to determining the percentage or amount of dry coal were alike for all coals. The ash, clinker and refuse from the ash-pits (11) in all instances was weighed back dry.

Coal which fell into the ashes from the firemen's shovels was picked out and burned, and, except during the first three or four tests, but few pieces of unburned or partially coked coal were found among the ashes.

The Wellston shaft coal, used during Tests Nos. 44 and 45, made a very light feathery ash, much of which was carried over the bridge walls by the draught through the fire boxes while the fires were being broken and cleaned.

The "water supplied to the boilers" (13) represents the water by meter, corrected for error, and reduced to weight for observed temperature of feed water (9). The "efficiency of the steam" (15) is obtained by dividing the average thermal value of the steam by calorimeter, by the total heat of saturated steam at the observed pressure. This takes into account the heat in the entrained water as well as that in the saturated steam, and credits the coal with the whole work done by the boilers; and since these tests were for relative coal effects, and not boiler effects, it seemed to the author entirely proper to credit each coal with the whole heat accounted for by the steam calorimeters.

The "net water converted into steam" (16) is the "water supplied to boilers" (13) multiplied by the "efficiency of the steam" (15). As the efficiency recognizes a small amount of heat in the "water entrained in the steam" (14), the term "net water converted into steam" is therefore not strictly correct.

In reducing "steam per pound of coal" (17) or "combustible"

(19) from "temperature of feed," to "steam per pound of coal" (18), or "combustible" (20), "from and at 212° Fahr.," the divisor 996 instead of 965.7 was used. The remaining captions will require no explanation.

While the physical results from the different coals are properly comparable with each other, they are not comparable with the results obtained from the same coals under other conditions of boilers, furnace, firemen, etc. (This remark, of course, does not apply to the proximate chemical analyses and determinations of thermal value, which will be referred to hereafter.)

The evaporative results from and at 212° Fahr. of Test No. 50 (Table No. 9), and Tests Nos. 51 to 55, inclusive (Table No. 10), have been corrected to agree with the data of Table No. 4, to correspond with the first forty-nine tests, all of which were made prior to the careful scaling of the boilers.

TABLE No. 5.—Pennsylvania Coals.

Kind of coal	Pittsburg—Brown, Second Pool.				Pittsburg—Catsburg, Third Pool.			
Grade of coal	Lump.	Barge Run.	Mine Run.	Nut & Slack	Lump.	Barge Run.	Mine Run.	Nut & Slack
Number of test	13	15	14	16	5	6	7	8
Date of test	Sept. 1, '97.	Sept. 8, '97.	Sept. 3, '97.	Sept. 10, '97.	Aug. 9, '97.	Aug. 11, '97.	Aug. 13, '97.	Aug. 16, '97.
1 Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2 Av. steam pressure, lbs	101.797	101.594	101.820	101.203	103.109	101.500	101.656	101.023
3 " depth of water over fire boxes, ins	5.582	5.767	5.541	5.480	6.906	6.983	6.672	6.211
4 " temp. of waste gases, Fahr	645.156°	608.950°	647.047°	609.470°	675.010°	736.150°	733.975°	707.930
5 " draft, gauge in chimney, ins	0.457	0.425	0.454	0.460	0.371	0.444	0.411	0.422
6 " temp. of air during test, Fahr	80.141°	81.187°	73.953°	82.040°	80.360°	75.580°	77.281°	72.094°
7 " dryness of air " " Fahr	7.657°	11.312°	9.984°	12.310°	9.340°	9.635°	12.336°	8.406°
8 " barometer " " ins	29.534	29.666	29.732	29.711	29.400	29.360	29.548	29.436
9 " temp. of feed water, Fahr	196.828°	196.856°	196.220°	194.780°	194.400°	193.547°	194.105°	196.210°
10 Commercially dry coal burned, lbs	20 166.00	20 087.00	21 413.00	21 623.00	19 441.00	20 805.00	21 345.00	21 823.00
11 Ash and clinker, weighed back dry, lbs	1 125.00	1 200.00	1 914.00	1 878.00	1 262.00	1 469.00	1 922.00	2 186.00
12 Percentage of non-combustible, %	5.579	5.989	8.937	8.685	6.491	7.061	9.004	10.017
13 Water supplied to boilers during test, lbs	173 117.301	173 150.484	171 861.891	173 637.407	170 561.758	173 805.131	172 922.832	171 080.318
14 Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15 Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16 Net water converted into steam, lbs	171 905.480	171 988.431	170 658.858	172 421.945	169 367.826	172 588.495	171 712.372	169 882.756
17 Steam per lb. of coal from temp. of feed, lbs	8.524	8.581	7.970	7.974	8.712	8.295	8.045	7.785
18 Steam per lb. of coal from and at 212° Fahr., lbs	9.004	9.064	8.424	8.439	9.227	8.790	8.521	8.227
19 Steam per lb. of combustible from temp. of feed, lbs	9.028	9.128	8.752	8.732	9.317	8.926	8.841	8.651
20 Steam per lb. of combustible from and at 212° Fahr., lbs	9.537	9.642	9.251	9.241	9.868	9.459	9.364	9.143
21 Steam per sq. foot of heating surface, per hour, lbs	5.397	5.398	5.358	5.413	5.317	5.419	5.391	5.334
22 Coal per sq. foot of grate surface, per hour, lbs	17.408	17.297	18.484	18.666	16.783	17.960	18.426	18.839
23 Relative efficiency on coal	0.881	0.887	0.825	0.826	0.903	0.860	0.834	0.805
24 Relative efficiency on combustible	0.885	0.895	0.859	0.858	0.916	0.878	0.869	0.849

TABLE No. 6.—Pennsylvania Coals.

	Kind of coal	Pittsburg—Cincinnati Third Pool.		Pittsburg—Woods, Fourth Pool.				
	Grade of coal	Mine Run.	Nut & Slack	Lump.	Barge Run	Mine Run.	Nut & Slack	
	Number of test	17	18	9	10	11	12	
	Date of test	Sept. 14, '97.	Sept. 16, '97.	Aug. 20, '97.	Aug. 23, '97.	Aug. 25, '97.	Aug. 27, '97.	
1	Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs. 6 m.	16 hrs.	
2	Av. steam pressure, lbs.	101.047	100.875	100.531	100.750	101.266	100.227	
3	" depth of water over fire boxes, ins.	5.607	5.717	5.779	5.876	5.562	5.234	
4	" temp. of waste gases, Fahr.	654.81°	631.656°	682.500°	679.897°	656.000°	661.453°	
5	" draft, gauge in chimney, ins.	0.441	0.455	0.427	0.450	0.451	0.444	
6	" temp. of air during test, Fahr.	83.508°	84.922°	71.797°	70.164°	70.141°	78.062°	
7	" dryness of air " " Fahr.	10.742°	10.281°	9.547°	5.188°	7.313°	9.046°	
8	" barometer " " ins.	29.657	29.572	29.584	29.469	29.444	29.448	
9	" temp. of feed water, Fahr.	194.852	196.150°	196.406°	195.206°	195.981°	196.960°	
10	Commercially dry coal burned, lbs.	20 839.00	20 922.00	20 707.00	21 352.00	20 508.00	21 393.00	
11	Ash and clinker, weighed back dry, lbs.	1 272.00	1 503.00	1 961.00	2 057.00	2 010.00	2 210.00	
12	Percentage of non-combustible, %	6.103	7.183	9.470	9.634	9.801	10.330	
13	Water supplied to boilers during test, lbs.	171 908.332	172 904.860	172 403.533	173 151.440	174 518.825	172 453.598	
14	Water entrained in the steam, corrected, %	0.965	0.965	0.965	0.965	0.965	0.965	
15	Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	
16	Net water converted into steam, lbs.	170 794.344	171 694.526	171 196.708	171 939.380	173 297.193	171 246.363	
17	Steam per lb. of coal from temp. of feed, lbs.	8.196	8.206	8.268	8.053	8.450	8.005	
18	Steam per lb. of coal from and at 212° Fahr., lbs.	8.673	8.672	8.735	8.519	8.933	8.452	
19	Steam per lb. of combustible from temp. of feed, lbs.	8.729	8.841	9.132	8.911	9.368	8.927	
20	Steam per lb. of combustible from and at 212° Fahr., lbs.	9.237	9.344	9.648	9.426	9.903	9.426	
21	Steam per sq. foot of heating surface, per hour, lbs.	5.302	5.391	5.375	5.398	5.407	5.377	
22	Coal per sq. foot of grate surface, per hour, lbs.	17.990	18.061	17.875	18.432	17 593	18.468	
23	Relative efficiency on coal	0.849	0.849	0.855	0.834	0.874	0.827	
24	Relative efficiency on combustible	0.858	0.867	0.896	0.875	0.919	0.875	

TABLE No. 7.—West Virginia Coals.

	Kind of coal	New River.				Loup Creek.			Pocahontas.	
	Grade of coal	Lump.	Lump.	Mine Run.	Mine Run.	Lump.	Mine Run.	Slack.	Lump.	Mine Run.
	Number of test	35	49	1	31	27	20	21	26	22
	Date of test	Nov. 18, '97.	Dec. 21, '97.	July 27, '97.	Nov. 8, '97.	Oct. 7, '97.	Sept. 20, '97.	Sept. 22, '97.	Oct. 5, '97.	Sept. 24, '97.
1	Duration of test	16 hrs.	16 hrs.	16 hrs. 10 m.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2	Av. steam pressure, lbs	101.336	100.906	100.485	101.387	101.566	102.289	98.289	101.402	101.500
3	Av. depth of water over fire boxes, ins	5.671	5.599	7.503	5.662	5.498	5.728	5.679	5.548	5.516
4	Av. temp. of waste gases, Fahr	622.656°	606.484°	631.300°	521.720°	637.562°	718.812°	637.828°	625.969°	632.719°
5	" draft, gauge in chimney, ins	0.496	0.486	0.375	0.421	0.468	0.527	0.460	0.443	0.460
6	" temp. of air during test, Fahr	42.027°	35.062°	84.250°	59.797°	60.687°	57.375°	62.484°	71.617°	68.922°
7	" dryness of air " " Fahr	4.930°	2.281°	9.110°	1.555°	8.296°	9.125°	9.765°	10.865°	11.250°
8	" barometer " " ins	29.935	29.487	29.400	29.153	29.582	29.806	29.621	29.523	29.620
9	" temp. of feed water, Fahr	192.172°	195.005°	191.210°	184.918°	196.222°	195.380°	195.536°	194.697°	194.940°
10	Commercially dry coal burned, lbs.	19 865.00	18 515.00	18 486.00	19 474.00	18 346.00	20 907.00	20 010.00	18 123.00	18 821.00
11	Ash and clinker, weighed back dry, lbs	1 394.00	795.00	1 608.00	841.00	719.00	851.00	955.00	1 102.00	1 318.00
12	Percentage of non-combustible, %	7.017	4.298	8.698	4.318	3.919	4.070	4.773	6.081	7.002
13	Water supplied to boilers during test, lbs	173 195.866	171 060.585	172 400.556	175 687.419	178 571.394	174 306.660	167 319.175	171 706.125	168 771.676
14	Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15	Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16	Net water converted into steam, lbs.	171 983.495	169 863.161	171 193.752	174 457.607	177 321.394	173 086.513	166 147.941	170 504.182	167 590.274
17	Steam per lb. of coal from temp. of feed, lbs	8.657	9.174	9.261	8.958	9.665	8.279	8.303	9.408	8.904
18	Steam per lb. of coal from and at 212° Fahr, lbs	9.186	9.707	9.834	9.572	10.215	8.759	8.776	9.958	9.423
19	Steam per lb. of combustible from temp. of feed, lbs	9.310	9.586	10.143	9.363	10.060	8.630	8.719	10.017	9.575
20	Steam per lb. of combustible from and at 212° Fahr., lbs	9.879	10.143	10.771	10.005	10.633	9.130	9.216	10.603	10.133
21	Steam per sq. foot of heating surface, per hour, lbs	5.400	5.333	5.319	5.477	5.567	5.434	5.216	5.353	5.262
22	Coal per sq. foot of grate surface, per hour, lbs	17.149	15.590	15.794	16.811	15.837	18.048	17.272	15.645	16.247
23	Relative efficiency on coal	0.899	0.950	0.963	0.937	1.000	0.857	0.859	0.975	0.922
24	Relative efficiency on combustible	0.917	0.942	1.000	0.929	0.987	0.848	0.856	0.984	0.941

TABLE No. 8.—West Virginia Coals.

Kind of coal	Kanawha Steam.	Kanawha Gas.	Acme.		Keystone.	Winifrede.		
Grade of coal	Lump.	Lump.	Mine Run.	Nut & Slack	Mine Run.	Lump.	Mine Run.	Nut & Slack
Number of test	2	3	32	33	34	23	24	25
Date of test	July 29, '97.	July 31, '97.	Nov. 10, '97.	Nov. 12, '97.	Nov. 15, '97.	Sept. 27, '97.	Sept. 29, '97	Oct. 1, '97.
1 Duration of test	16 hrs. 11 m.	16 hrs. 1½ m.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2 Av. steam pressure, lbs	102.934	102.370	101.812	102.195	102.227	102.387	102.262	101.809
3 " depth of water over fire boxes, ins	7.322	6.856	5.620	5.682	5.668	5.533	5.565	5.399
4 " temp. of waste gases, Fahr	707.910°	696.453°	545.547°	539.766°	566.250°	541.984°	601.047°	575.969°
5 " draft, gauge in chimney, ins	0.406	0.392	0.488	0.487	0.487	0.420	0.445	0.434
6 " temp. of air during test, Fahr	85.530°	84.500°	50.812°	42.687°	64.820°	65.719°	69.422°	72.828°
7 " dryness of air " " Fahr	11.430°	9.330°	5.117°	6.437°	3.047°	11.672°	12.547°	12.125°
8 " barometer " " ins	29.530	29.570	29.399	29.733	29.569	29.780	29.774	29.762
9 " temp. of feed water, Fahr	188.870°	186.600°	193.640°	194.514°	191.700°	195.658°	195.184°	194.742°
10 Commercially dry coal burned, lbs	19 818.00	21 166.00	19 244.00	20 415.00	20 987.00	19 741.00	21 061.00	22 176.00
11 Ash and clinker, weighed back dry, lbs	1 322.00	2 653.00	1 171.00	1 786.00	1 886.00	1 390.00	1 190.00	2 874.00
12 Percentage of non-combustible, %	6.671	12.534	6.085	8.748	8.986	7.041	5.693	12.960
13 Water supplied to boilers during test, lbs	173 095.016	172 178.221	174 942.850	176 368.465	175 528.289	173 131.799	172 244.013	175 974.437
14 Water entrained in the steam, corrected, %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15 Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16 Net water converted into steam, lbs	171 883.351	170 972.973	173 718.250	175 133.886	174 293.591	171 919.876	170 998.585	174 742.616
17 Steam per lb. of coal from temp. of feed, lbs	8.673	8.078	9.027	8.579	8.305	8.709	8.119	7.880
18 Steam per lb. of coal from and at 212° Fahr., lbs	9.280	8.620	9.566	9.066	8.818	9.211	8.591	8.341
19 Steam per lb. of combustible from temp. of feed, lbs	9.293	9.235	9.612	9.401	9.125	9.368	8.609	9.053
20 Steam per lb. of combustible from and at 212° Fahr., lbs	9.943	9.854	10.185	9.935	9.688	9.908	9.110	9.583
21 Steam per sq. ft. of heating surface, per hour, lbs	5.335	5.340	5.454	5.499	5.472	5.398	5.369	5.486
22 Coal per sq. ft. of grate surface, per hour, lbs	16.914	18.262	16.613	17.623	18.118	17.041	18.181	19.144
23 Relative efficiency on coal	0.908	0.844	0.936	0.887	0.863	0.902	0.841	0.816
24 Relative efficiency on combustible	0.923	0.915	0.946	0.922	0.899	0.920	0.846	0.890

TABLE No. 9.—West Virginia Coals.

Kind of coal	Thacker.				Logan Cons. Thacker.	Maritime Thacker.	Campbell's Creek.	
Grade of coal	Lump.	Mine Run.	Mine Run.	Nut & Slack	Mine Run.	Mine Run.	Mine Run.	Nut & Slack
Number of test	37	28	50	30	42	46	22	19
Date of test	Nov. 22, '97.	Oct. 11, '97.	Dec. 30, '97.	Oct. 15, '97.	Dec. 4, '97.	Dec. 13,'97.	Oct. 13, '97.	Sept. 18,'97.
1 Duration of test	15 hrs. 30 m.	16 hrs.	15 hrs. 41m.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2 Av. steam pressure, lbs.	101.919	101.766	101.375	101.504	102.281	101.602	101.836	102.000
3 " depth of water over fire boxes, ins.	5.663	5.431	5.266	5.443	5.568	5.701	5.515	5.571
4 " temp. of waste gases, Fahr.	544.274°	567.281°	456.094°	589.516°	590.547°	568.047°	589.406°	646.187°
5 " draft, gauge in chimney, ins.	0.458	0.432	0.421	0.452	0.465	0.470	0.445	0.453
6 " temp. of air during test, Fahr.	47.194°	70.156°	43.062°	76.281°	41.773°	45.219°	70.914°	65.109°
7 " dryness of air " " Fahr.	2.726°	4.836°	2.539°	9.953°	1.876°	2.367°	9.234°	8.531°
8 " barometer " " ins.	29.567	29.187	29.417	29.808	29.391	29.272	29.801	29.518
9 " temp. of feed water, Fahr.	193.456°	195.892°	192.699°	197.240°	196.667°	194.442°	195.659°	195.787°
10 Commercially dry coal burned, lbs.	18 894.00	20 074.00	19 063.00	20 310.00	19 628.00	19 851.00	20 634.00	22 871.00
11 Ash and clinker, weighed back dry, lbs.	1 028.00	1 464.00	1 145.00	2 174.00	1 294.00	851.00	1 570.00	2 078.00
12 Percentage of non-combustible, %	5.467	7.293	6.007	10.704	6.592	4.287	7.609	9.085
13 Water supplied to boilers during test, lbs.	169 077.320	179 581.271	176 216.941	175 119.744	171 325.522	174 899.655	175 336.942	172 087.652
14 Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15 Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16 Net water converted into steam, lbs.	167 893.779	178 324.202	174 983.422	173 893.906	170 126.243	173 675.357	174 104.618	170 833.388
17 Steam per lb. of coal from temp. of feed, lbs.	8.929	8.883	9.179	8.562	8.667	8.749	8.438	7.469
18 Steam per lb. of coal from and at 212° Fahr., lbs.	9.464	9.392	9.735 (9.521)	9.040	9.158	9.264	8.924	7.899
19 Steam per lb. of combustible from temp. of feed, lbs.	9.445	9.582	9.766	9.588	9.279	9.141	9.133	8.216
20 Steam per lb. of combustible from and at 212 Fahr. lbs.	10.011	10.132	10.357 (10.129)	10.124	9.804	9.670	9.659	8.689
21 Steam per sq. ft. of heating surface, per hr., lbs. (Corrected)	5.441	5.599	5.605 (5.482)	5.460	5.341	5.453	5.466	5.364
22 Coal per sq. ft. of grate surface, per hour., lbs.	16.756	17.329	16.677	17.533	16.944	17.137	17.812	19.744
23 Relative efficiency on coal	0.926	0.919	0.932	0.885	0.896	0.907	0.874	0.773
24 Relative efficiency on combustible	0.929	0.941	0.940	0.910	0.910	0.899	0.897	0.807

TABLE No. 10.—West Virginia Coals.

	Kind of coal	Monongah.	Eagle.	Cedar Grove.	Mt. Carbon.	Chesapeake.	Coalburg.	Belmont Splint.	
	Grade of coal	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Mine Run.	Nut & Slack
	Number of test	36	51	52	53	54	55	38	39
	Date of test	Nov. 20, '97.	Mar. 29, '98.	Mar. 31, '98.	April 2, '98.	April 5, '98.	April 7, '98.	Nov. 24, '97.	Nov. 26, '97.
1	Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2	Av. steam pressure, lbs	101.625	97.949	99.254	100.308	100.629	100.441	102.004	101.406
3	" depth of water over fire boxes, ins	5.736	5.551	5.478	5.701	5.711	5.672	5.744	5.719
4	" temp. of waste gases, Fahr	548.984°	436.094°	531.094°	549.844°	539.690°	551.484°	570.781°	585.938°
5	" draft, gauge in chimney, ins	0.424	0.390	0.458	0.458	0.458	0.454	0.495	0.462
6	" temp. of air during test, Fahr	58.406°	45.961°	41.406°	47.672	34.549°	46.047°	41.531°	55.047°
7	" dryness of air " " Fahr	7.844°	5.844°	4.945°	6.984°	5.625°	6.985°	4.984°	2.578°
8	" barometer " " ins	29.523	29.875	29.788	29.651	29.709	29.841	29.801	29.441
9	" temp. of feed water, Fahr	193.421°	198.389°	196.432°	194.630	192.549	195.304	194.627°	193.516°
10	Commercially dry coal burned, lbs	20 479.00	19 307.00	18 595.00	19 464.00	19 626.00	19 506.00	21 174.00	22 516.00
11	Ash and clinker, weighed back dry, lbs	1 375.00	1 622.00	898.00	1 331.00	1 075.00	1 376.00	1 661.00	2 722.00
12	Percentage of non-combustible, %	6.714	8.401	4.829	6.838	5.477	7.054	7.845	12.089
13	Water supplied to boilers during test, lbs	173 943.320	170 602.293	169 779.607	168 968.583	167 985.909	166 936.184	176 043.283	174 437.084
14	Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15	Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16	Net water converted into steam, lbs	172 725.717	169 408.077	168 591.150	167 785.803	166 809.999	164 873.981	174 810.980	173 216.024
17	Steam per lb. of coal from temp. of feed, lbs	8.434	8.774	9.066	8.620	8.499	8.452	8.256	7.693
18	Steam per lb. of coal from and at 212° Fahr., lbs.	8.939	9.248	9.576	9.121	9.014	8.939	8.730	8.152
	(Corrected)		(9.044)	(9.365)	(8.920)	(8.816)	(8.742)		
19	Steam per lb. of combustible from temp. of feed, lbs	9.041	9.579	9.527	9.253	8.992	9.094	8.959	8.751
20	Steam per lb. of combustible from and at 212° Fahr., lbs	9.582	10.096	10.062	9.791	9.536	9.618	9.485	9.273
	(Corrected)		(9.874)	(9.841)	(9.576)	(9.326)	(9.406)		
21	Steam per sq. foot of heating surface, per hr., lbs.	5.423	5.319	5.293	5.268	5.237	5.176	5.488	5.438
	(Corrected)		(5.202)	(5.177)	(5.152)	(5.122)	(5.062)		
22	Coal per sq. foot of grate surface, per hr., lbs	17.679	16.667	16.052	16.802	16.942	16.839	18.278	19.437
23	Relative efficiency on coal	0.875	0.885	0.917	0.873	0.863	0.856	0.855	0.799
24	Relative efficiency on combustible	0.890	0.917	0.914	0.889	0.866	0.873	0.881	0.861

TABLE No. 11.—Ohio, Kentucky and Tennessee Coals.

Kind of coal	Luhrig-Washed.		Wellston-Shaft.		Jellico—Kentucky.		Jellico-Tenn.
Grade of coal	Egg.	Slack.	Mine Run.	Nut& Slack	Mine Run.	Nut& Slack	Mine Run.
Number of test	41	43	44	45	47	40	48
Date of test	Dec. 1, '97.	Dec. 6, '97.	Dec. 8, '97.	Dec. 10, '97.	Dec. 15, '97.	Nov. 20, '97.	Dec. 17, '97.
1 Duration of test	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.	16 hrs.
2 Av. steam pressure, lbs	102.246	101.539	101.281	100.867	101.454	102.234	101.484
3 " depth of water over fire boxes, ins	5.676	5.588	5.549	5.689	5.521	5.830	5.528
4 " temp. of waste gases, Fahr	565.860°	642.578	650.937°	652.188°	613.906°	581.250°	633.281°
5 " draft, gauge in chimney, ins	0.479	0.479	0.484	0.479	0.497	0.506	0.499
6 " temp. of air during test, Fahr	37.125°	39.352°	52.344°	61.516°	42.914°	34.234°	38.414°
7 " dryness of air " " Fahr	4.250°	2.493°	3.172°	5.103°	2.875°	3.867°	1.797°
8 " barometer " " ins	29.606	20.479	29.514	29.482	29.512	29.820	29.667
9 " temp. of feed water, Fahr	192.606°	196.351°	195.911	195.176°	194.924°	194.160°	197.505°
10 Commercially dry coal burned, lbs	23 211.00	24 541.00	22 251.00	23 475.00	20 283.00	23 159.00	21 147.00
11 Ash and clinker, weighed back dry, lb	1 611.00	1 255.00	665.00	1 104.00	680.00	2 441.00	602.00
12 Percentage of non-combustible, %	6.941	5.114	2.989	4.703	3.352	10.540	3.130
13 Water supplied to boilers during test, lbs	172 327.398	170 140.303	170 685.354	169 160.625	175 926.554	171 270.734	175 901.457
14 Water entrained in the steam, corrected %	0.965	0.965	0.965	0.965	0.965	0.965	0.965
15 Efficiency of the steam, corrected	0.993	0.993	0.993	0.993	0.993	0.993	0.993
16 Net water converted into steam, lbs	171 121.047	168 949.321	169 490.556	167 976.501	174 695.068	170 071.839	174 670.147
17 Steam per lb. of coal from temp of feed, lbs	7.372	6.884	7.617	7.156	8.613	7.344	8.260
18 Steam per lb. of coal from and at 212°, Fahr., lbs	7.820	7.275	8.053	7.570	9.115	7.779	8.719
19 Steam per lb. of combustible from temp. of feed, lbs	7.922	7.255	7.852	7.509	8.912	8.209	8.527
20 Steam per lb. of combustible from and at 212° Fahr., lbs	8.404	7.667	8.301	7.943	9.431	8.695	9.001
21 Steam per sq. foot of heating surface, per hour, lbs	5.373	5.304	5.321	5.274	5.485	5.340	5.484
22 Coal per sq. foot of grate surface, per hour, lbs	20.037	21.186	19.209	20.265	17.509	19.991	18.255
23 Relative efficiency on coal	0.765	0.712	0.788	0.741	0.892	0.761	0.853
24 Relative efficiency on combustible	0.780	0.712	0.771	0.737	0.876	0.807	0.836

The relative efficiency of the several coals, on coal (23), and on combustible (24) are based upon the highest evaporation obtained during the tests per pound of coal or combustible from and at 212° Fahr. as standards, and the factors for multiplying the prices at which the several coals may be offered are, of course, the reciprocals of these efficiencies. Thus, if the coals mentioned in Table No. 12 were offered at the prices per ton annexed, then column 3 contains the deduced prices, obtained in the manner described.

TABLE No. 12.

Kind of coal.	Dealers' price per ton.	Deduced price for comparison of bids.
Pittsburg, Brown, Second Pool, Mine Run	$1.50	$1.82
Pittsburg, Catsburg, Third Pool, Mine Run	1.75	2.10
Pittsburg, Woods, Fourth Pool, Mine Run	1.60	1.83
Pocahontas, Mine Run	1.75	1.90
New River, Mine Run	1.64	1.73
Wellston Shaft, Mine Run	1.90	2.41

Upon these prices, New River coal would be offered at the lowest cost to the steam user.

The reciprocals of the efficiencies based on coal also show how much more than the standard will be required of any of the coals tested to do a given amount of work.

The economic and capacity results of the Pennsylvania coals are shown in Table No. 13.

The relative values of the grades of Second Pool coal noted in the table, from the dealers' point of view, are about as follows:

Lump	100
Barge Run	90–95
Mine Run	80–90
Nut and Slack	70–80

The grades of coal show differences in value to the buyer or steam user as follows:

Lump	100.00
Barge Run	100.67
Mine Run	93.56
Nut and Slack	93 72

TABLE No. 13.—Economic and Capacity Results with Pennsylvania Coals.

Test numb	Kind of coal.	Steam from and at 212° Fahr.		Steam per square foot of heating surface per hour.	Coal per square foot of grate surface per hour.
		Per pound of coal.	Per pound of combustible.		
		Pounds.	Pounds.	Pounds.	Pounds.
13..	Pittsburg, Brown, Second Pool, Lump.....	9.004	9.537	5.397	17.408
15..	" " " Barge Run..	9.064	9.642	5.398	17.297
14..	" " " Mine Run...	8.424	9.251	5.358	18.484
16..	" " " Nut and Slack.....	8.439	9.241	5.413	18.666
5..	Pittsburg, Catsburg, Third Pool, Lump.....	9.227	9.868	5.317	16.783
6..	" " " Barge Run	8.790	9.459	5.419	17.960
7..	" " " Mine Run.	8.521	9.364	5.391	18.426
8..	" " " Nut and Slack....	8.227	9.143	5.334	18.839
17..	Pittsburg, Cincinnati, Third Pool, Mine Run	8.673	9.237	5.362	17.990
18..	" " " Nut and Slack....	8.672	9.344	5.391	18.061
9..	Pittsburg, Woods, Fourth Pool, Lump.....	8.735	9.648	5.375	17.875
10..	" " " Barge Run.	8.519	9.436	5.398	18.432
11..	" " " Mine Run..	8.933	9.903	5.407	17.593
12..	" " " Nut and Slack....	8.452	9.426	5.377	18.468

With this coal it appeared that there was no superiority of Lump over Barge Run coal, while Mine Run and Nut and Slack were less than $6\frac{1}{2}\%$ inferior in economy to Lump coal; and Nut and Slack coal for steam purposes, at the prices usually asked, was superior to Lump coal by 26.20%; or while one dollar's worth of Second Pool, Lump coal furnished 10 290 lbs. of steam, one dollar's worth of Nut and Slack coal furnished 12 983 lbs. of steam, and one dollar's worth of Mine Run coal furnished 11 232 lbs. of steam.

The Third Pool coals, for the different grades, have the following relative mean values:

Lump	100.00
Barge Run	92.35
Mine Run	93.17
Nut and Slack	92.12

The Fourth Pool coals have the following values for the different grades:

Lump	100.00
Barge Run	97.53
Mine Run	102.27
Nut and Slack	96.76

TABLE No. 14.—Economic and Capacity Results with West Virginia Coals.

Test No.	Kind of coal.	Steam from and at 212° Fahr.		Steam per square foot of heating surface per hour.	Coal per square foot of grate surface per hour.
		Per pound of coal.	Per pound of combustible.		
		Pounds.	Pounds.	Pounds.	Pounds.
35....	New River, Lump	9.186	9,879	5,400	17.149
49....	" Lump	9.707	10,143	5,333	15.590
1....	" Mine Run	9.834	10,771	5,319	15.794
31....	" Mine Run	9.572	10,065	5,477	16.811
27....	Loup Creek, Lump*	10.215	10,633	5,567	15.837
20....	" Mine Run†	8.759	9,130	5,434	18.048
21....	" Slack*	8.776	9,216	5,216	17.272
26....	Pocahontas, Lump	9.958	10,603	5,353	15.645
22....	" Mine Run	9.423	10,133	5,262	16.247
2....	Kanawha, Steam Lump	9.280	9,943	5,335	16.914
3....	" Gas Lump	8.620	9,854	5,340	18.262
32....	Acme, Mine Run	9.566	10,185	5,454	16.613
33....	" Nut and Slack	9.066	9,935	5,499	17.623
34....	Keystone, Mine Run	8.818	9,688	5,472	18.118
23....	Winifrede, Lump	9.211	9,908	5,398	17.041
24....	" Mine Run	8.591	9,110	5,393	18.181
25....	" Nut and Slack	8.341	9,583	5,486	19.144
37....	Thacker, Lump	9.464	10,011	5,441	16.756
28....	" Mine Run	9.392	10,132	5,599	17.329
50....	" Mine Run	9.521	10,129	5,482	16.677
30....	" Nut and Slack	9.040	10,124	5,460	17.533
42....	Logan Consolidated, Thacker, Mine Run	9.158	9,804	5,341	16.944
46....	Maritime, Thacker, Mine Run	9.264	9,679	5,458	17.137
29....	Campbell's Creek, Mine Run	8.924	9,659	5,466	17.812
19....	" " Nut and Slack	7.899	8,689	5,364	19.744
36....	Monongah, Mine Run	8.939	9,582	5,423	17.679
51....	Eagle, Mine Run	9.044	9,874	5,202	16.667
52....	Cedar Grove, Mine Run	9.365	9,841	5,177	16.052
53....	Mount Carbon, Mine Run	8.930	9,576	5,152	16.802
54....	Chesapeake, Mine Run	8.816	9,326	5,122	16.942
55....	Coalburg, Mine Run	8.742	9,406	5,062	16.839
38....	Belmont Splint, Mine Run	8.730	9,485	5,488	18.278
39....	" " Nut and Slack	8.152	9,273	5,438	19.437

* From the MacDonald Mine.
† From the Dunn Loup Mine.

An examination of these figures shows two things:

First.—That the opinion hitherto generally accepted as to Second Pool, Monongahela River coal being markedly superior to the coals obtained from the newer Pittsburg mines farther up the river, is not sustained by the results of these tests.

Second.—That the prices usually asked for the higher grades of these coals are not warranted for steam purposes; that Mine Run and Nut and Slack coals are by far the most economical coals, and that they can be sold upon narrower discounts from Lump coal prices and still be the most profitable to buyers of steam coals.

Referring briefly to the economic results per pound of combustible, it will be noticed that there is greater uniformity among all these coals in the steam per pound of combustible, without regard to the mines or grades, than in the steam per pound of coal, and this naturally is due to the fact that the lower grades contain just as good coal as Lump, but carry more waste matter from the slate and stone loosened in mining from the roof and floor of the galleries, which in Lump coal is presumed to be removed when it is run over the screen at the mouth of the mine, and by the use of finer screens often goes into the cargoes of low-grade coals, and is purchased as so much coal by the buyer. Assuming no more non-combustible matter in the Mine Run than in the Lump coal, such coal for steam purposes should always be equal to Lump, provided the grates upon which it is burned are suitable to retain the smaller and finer particles until coking occurs, or in the non-coking coals, until combustion is completed.

A review of the Pittsburg coals indicates that they should be rated as of equal quality, whether obtained from the older mines in the Second Pool or from the newer mines in the Fourth Pool.

The economic and capacity results of the West Virginia coals are shown in Table No. 14.

The economic and capacity results of the Ohio, Kentucky and Tennessee coals are shown in Table No. 15.

TABLE No. 15.—Economic and Capacity Results with Ohio, Kentucky and Tennessee Coals.

Test No.	Kind of coal.	Steam from and at 212° Fahr.		Steam per square foot of heating surface per hour.	Coal per square foot of grate surface per hour.
		Per pound of coal.	Per pound of combustible.		
		Pounds.	Pounds.	Pounds.	Pounds.
	Ohio Coals.				
41	Luhrig, Washed Egg	7.820	8.404	5.373	20.037
43	Luhrig, Washed Slack	7.375	7.667	5.304	21.186
44	Wellston Shaft, Mine Run	8.053	8.301	5.321	19.369
45	Wellston Shaft, Nut and Slack	7.570	7.943	5.274	20.265
	Kentucky Coals.				
47	Jellico, Mine Run	9.115	9.431	5.485	17.509
40	Jellico, Nut and Slack	7.779	8.695	5.340	19.991
	Tennessee Coal.				
48	Jellico, Mine Run	8.719	9.001	5.484	18.255

A hasty glance at the table of economic results obtained with the West Virginia coals will show that nearly all the samples should be ranked higher than the Pittsburg coals. Thus the average weight of steam per pound of coal for all the Pittsburg coals was 8.69 lbs., an economy which is reached or excelled by all except five samples of West Virginia coals. Heretofore, in the western river markets, Pittsburg coal from the Second Pool has been the accepted standard, and for years prices and guarantees have been based on this coal.

Considering Pittsburg coal as the standard for steam purposes in the western river markets, all but five of the West Virginia coals exceed the average of the Pittsburg coals in steam per pound of coal and per pound of combustible. Belmont Splint, Winifrede, Campbell's Creek, Kanawha Gas, and the Loup Creek low-grade coals are the only kinds or grades that fall below the Pittsburg coals. Of the West Virginia coals, several, like New River, Pocahontas, Loup Creek (from the MacDonald Mine), Acme, Thacker, Cedar Grove and Kanawha Steam, are peculiarly of good quality.

Grouping the West Virginia coals in two classes, of which the first contains coals distinctly superior to Pittsburg, and the second all other coals tested, the average performance for the coals of each of these classes is shown in Table No. 16.

TABLE No. 16.

Samples of coals.	Steam from and at 212° Fahr.	
	Per pound of coal. Pounds.	Per pound of combustible. Pounds.
Pittsburg coals (average)	8.69	9.46
First-class, West Virginia coals: New River, Loup Creek (MacDonald Mine), Pocahontas, Kanawha Steam, Acme, Thacker, and Cedar Grove coals (average)	9.51	10.16
Second-class, West Virginia coals: Loup Creek (Dunn Loup Mine), Kanawha Gas, Keystone, Winifrede, Logan Consolidated Thacker, Maritime Thacker, Campbell's Creek, Monongah, Eagle, Mt. Carbon, Chesapeake, Coalburg and Belmont Splint coals (average)	8.76	9.49

By referring to the proximate analyses and calorific tests of the seven coals grouped in the first class, it will be noticed that the composition of the combustible matter varies widely, while the heating

powers of combustible and practical performance are sufficiently near together to suggest coals of like character.

TABLE No. 17.

Sample of coal.	B. T. U. per pound of combustible.	Steam per pound of combustible (averages).
New River	15 757	10.20
Loup Creek (MacDonald Mine)	15 336	10.63
Pocahontas	15 456	10.37
Kanawha Steam	15 653	9.94
Acme	15 589	10.06
Thacker	15 554	10.10
Cedar Grove	15 422	9.84

From the proximate analyses of these coals, the averages of fixed carbon and volatile matter are as follows:

Sample of coal.	Fixed carbon.	Volatile matter.
New River, Loup Creek and Pocahontas	78.77	18.85
Kanawha Steam, Acme and Thacker	64.72	31.37
Cedar Grove	60.26	35.35

Referring to the Ohio, Kentucky and Tennessee coals tested, it is apparent that for steam purposes these coals are not equal to the West Virginia and Pittsburg coals. (An exception should be made of the Jellico, Ky., coal, which compares favorably with the second-class West Virginia coals.)

The diagrams (Figs. 3 to 6, inclusive) show the relation of the different coals of each group and grade. The vertical scale represents pounds, and the horizontal scale the different coals. The solid circles connected by lines at the top of the diagrams indicate (to scale) the average weights of coal samples burned per square foot of grate surface per hour. The second series of circles connected by lines indicates the steam per pound of combustible; and the third series of circles connected by lines indicates the steam per pound of coal, both from and at 212° Fahr. The lower solid circles and lines indicate the steam in pounds per square foot of heating surface per hour. The reasonable uniformity of the work done during each test by all coals will be noticed upon reference to the several diagrams.

It is quite possible that the boilers used in these tests were not the best for all the coals tested, and that some kinds and grades of coal

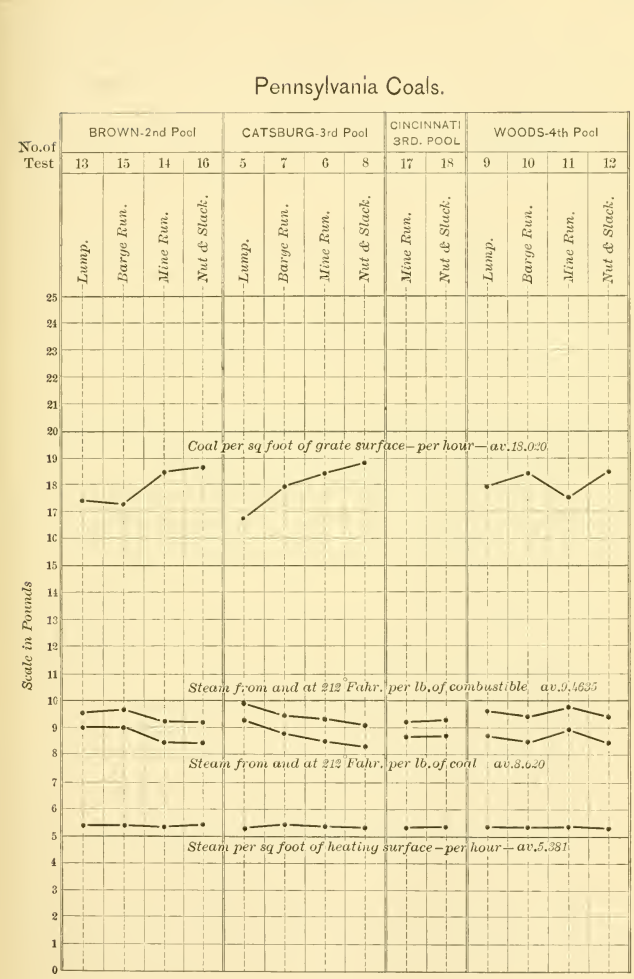

Pennsylvania Coals.
BROWN-2nd Pool
CATSBURG-3rd Pool
CINCINNATI 3RD. POOL
WOODS-4th Pool
No.of Test
13 15 14 16 5 7 6 8 17 18 9 10 11 12
Lump. Barge Run. Mine Run. Nut & Slack. Lump. Barge Run. Mine Run. Nut & Slack. Mine Run. Nut & Slack. Lump. Barge Run. Mine Run. Nut & Slack.
Coal per sq foot of grate surface–per hour—av.13.020
Steam from and at 212 Fahr. per lb. of combustible av.9.4635
Steam from and at 212 Fahr. per lb. of coal av.8.620
Steam per sq foot of heating surface–per hour—av.5.381
Scale in Pounds
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

FIG. 3.

suffered accordingly; but that is a condition which perhaps will always be found true, for no form of boiler and furnace can be equally well adapted to every variety of bituminous coal, nor could changes be made in the boiler and furnace, and in the mode of firing, for different kinds of coal, without raising a suspicion in the investigator's mind that by so doing he would be favoring one coal at the expense of another.

The Galloway boilers have elsewhere been reported upon very favorably when worked with bituminous coal, results being attributed to them, with what the author finds to be inferior coals, which considerably exceed the performance of any coal tested during this investigation, but doubtless such results were obtained under conditions which cannot be had in the daily operation of steam boilers with ordinary firemen.

These boilers were originally set with 960 sq. ft. of heating surface and 199 sq. ft. of super-heating surface, a total combined heating and super-heating surface of 1 159 sq. ft. for each boiler. The grate surface, of course, was the same as during the author's trials, but the chimney was originally 150 ft. high and of the same section as during the later tests. The report of the contract trial upon which the boilers were accepted by the city contains the following data:

Date of trial	May, 1889.
Duration of trial	10 hrs.
Steam pressure, by gauge	92.80 lbs.
Temperature of feed water	193.70° Fahr.
Temperature of waste gases in flue leading to chimney*	700° + Fahr.
Dry coal consumed	6 911.00 lbs.
Super-heat, expressed as percentage gain of heat units above saturated steam (no data given)	11.03%
Total steam from and at 212° Fahr.	76 377.00 lbs.
Steam per pound of dry coal from and at 212° Fahr.	11.05 lbs.

The coal for the contract trial is stated as Pittsburg Nut, containing 10.3% of ash, etc., and over 3½% of moisture. These percentages are much larger than found by the author for any kind or grade of Pittsburg coal. The coal was burned at the rate of 19.05 lbs. per square

* Obtained by melting zinc.

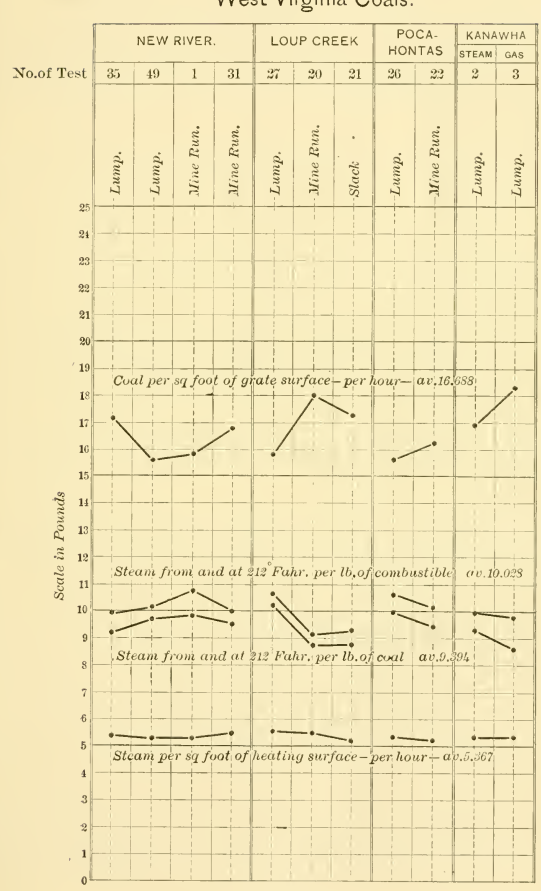

West Virginia Coals.
NEW RIVER.
LOUP CREEK
POCA-HONTAS
KANAWHA
STEAM
GAS
No.of Test
35
49
1
31
37
20
21
26
22
2
3
Lump.
Lump.
Mine Run.
Mine Run.
Lump.
Mine Run.
Slack
Lump.
Mine Run.
Lump.
Lump.
Coal per sq foot of grate surface—per hour— av.16.688
Steam from and at 212° Fahr. per lb. of combustible av.10.023
Steam from and at 212 Fahr. per lb. of coal av.9.394
Steam per sq foot of heating surface—per hour— av.5.367
Scale in Pounds
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25

FIG. 4.

foot of grate surface per hour, and steam was made at the rate of 8.03 lbs. from and at 212° Fahr. per square foot of water-heating surface per hour. The principal differences between the conditions for the contract trial, and those for the coal tests by the author, were in the setting of the boilers and in the height of the chimney, with the complete loss of super-heating surface for the later trials, which apparently alone made the efficiency of the steam 11.73% better for this earlier trial.

The author has no record of the methods pursued, and no means of checking the results of the contract trial, but it would be interesting to know how the very high percentage of super-heat was obtained with a boiler setting which within his experience has seldom shown above a few degrees of super-heat, and generally is expected by the builders only to secure dry steam, and further, how the largely increased capacity was obtained. During the author's tests the boilers were supposed to be doing the best possible work, with an economic rate of coal consumption per square foot of grate surface per hour. If there was error in reporting the contract trial, it would seem to be either in the weight of water credited to the boiler, or in the quality of steam made, or both. The rate of coal consumption compares fairly well with the rates used during the author's coal tests.

In all instances the coals were used with care, and the furnace conditions were such as, in the author's opinion, would give the coal under trial a fair rating. One circumstance, however, seemed to militate against all coals, *i. e.*, the very high temperature of the chimney gases, which suggested at times that the coals were being burned at too high a rate per unit of grate surface, or that the ratio of grate and heating surface was not well proportioned. The higher temperatures of hot gas were obtained during the earlier tests, and this was then thought to have been due to the fireman's lack of previous experience with these boilers, but singularly enough some of the high temperatures in the chimney were accompanied by better economies than when the same coals were tried with lower temperatures of waste gases. By referring to the abstract from the reported results of the contract trial, it seems that the waste gases entered the chimney then at a very high temperature, notwithstanding the unusual economic results obtained.

During the first 22 tests there was no regulation of the flow of air to

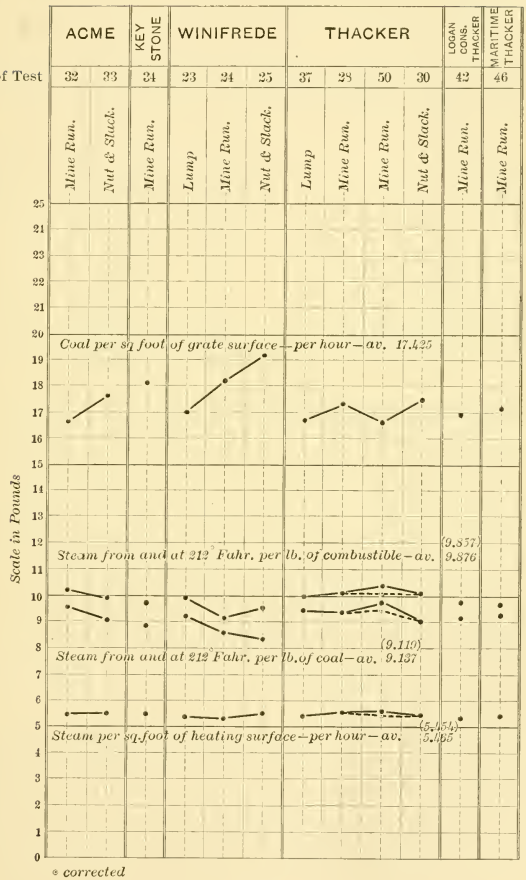

West Virginia Coals.
ACME
KEY STONE
WINIFREDE
THACKER
LOGAN CONS. THACKER
MARITIME THACKER
No.of Test
32
33
34
23
24
25
37
28
30
42
46
Mine Run.
Nut & Slack.
Mine Run.
Lump
Mine Run.
Nut & Slack.
Lump
Mine Run.
Mine Run.
Nut & Slack.
Mine Run.
Mine Run.
Scale in Pounds
Coal per sq.foot of grate surface—per hour—av. 17.425
Steam from and at 212° Fahr. per lb. of combustible—av.
(9.877)
9.876
Steam from and at 212° Fahr. per lb. of coal—av. 9.137
(9.119)
Steam per sq.foot of heating surface—per hour—av.
(5.454)
5.455
corrected

FIG. 5.

the ash-pits, and comparison of coals of like kinds tried before and after the quantity of air to the ash-pits was placed under control, indicates in one example a gain, and in the other a slight loss by the change.

TABLE No. 18.—Comparison of Coals tried before and after Regulation of Air to Ashpits.

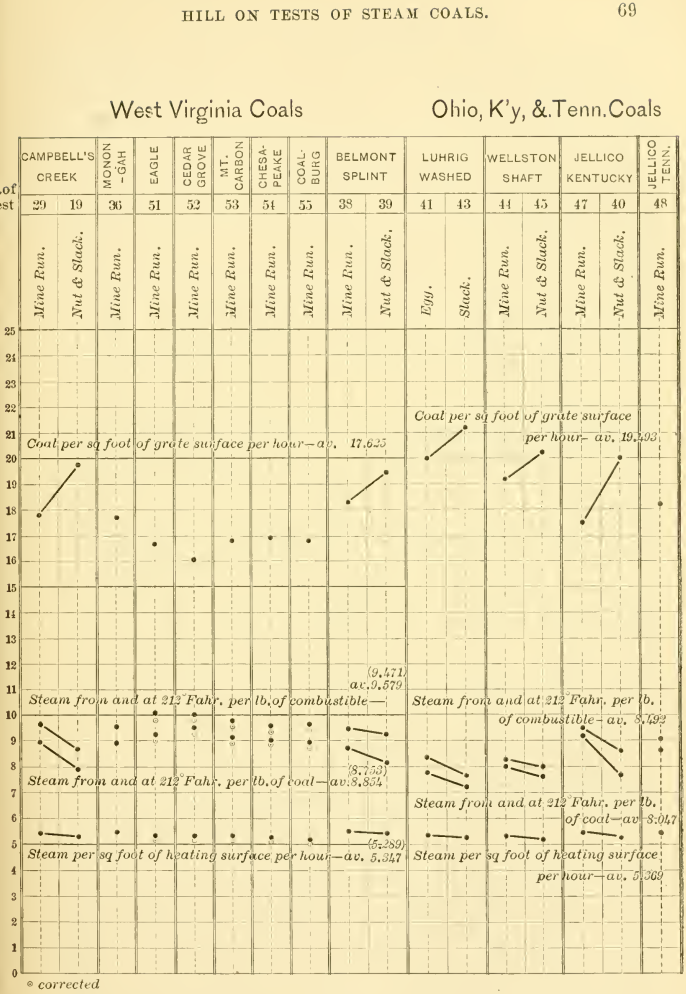
Test Number.	Kind of coal.	Steam from and at 212° Fahr.		Temperature of waste gases.
		Per pound of coal. Pounds.	Per pound of combustible. Pounds.	Fahr.
1	New River, Mine Run.....	9.834	10.771	631.3
20	Loup Creek, Mine Run.....	8.759	9.130	718.8
31	New River, Mine Run	9.572	10.005	521.7
35	New River, Lump.....	9.186	9.879	622.6
49	New River, Lump.....	9.707	10.143	606.5
27	Loup Creek, Lump.....	10.215	10.633	637.6

Calorimetric Determinations of Heating Power of Coals.

Before proceeding to this branch of the investigation, careful study was made of several modern forms of coal calorimeter, and, after weighing the advantages and disadvantages of each, the author decided to use the instrument devised by Professor R. C. Carpenter, of Cornell University, the accuracy of which, after observing due precautions in its use, depends upon the assumption that equal increments of heat produce corresponding dilatations of the water in the instrument, and that the heat value for pure carbon has been determined. The instrument was accompanied by a diagram, which gave the manufacturer's calibration and value for each inch of rise on the scale.

By burning in the instrument successive weights of pure carbon, obtained by coking recrystallized sugar, the value in heat units of 1 in. on the scale of the water column was obtained, and by multiplying this by the rise in inches and dividing by the weight of fuel burned in fractions of a pound, the heat units per pound of coal or coke sample were obtained. The construction and use of this form of coal calorimeter is fully described in a paper by Professor Carpenter.*

* *Transactions*, A. S. M. E., Vol. xvi, 1895.



* corrected

FIG. 6.

The value of pure carbon burning without a residue, as determined by Berthelot, is 14 652 B. T. U. per pound,* while two samples of pure carbon, burning without a residue in the crucible, gave during the author's tests a mean of 14 646 when measured on the heat scale supplied with the Carpenter instrument.

The preliminary tests with coke from sugar were carefully repeated with trifling variations in the results, and, accepting the value of pure carbon as determined by Berthelot, then 1 in. rise of the water column equals 7.15 B. T. U.; *i. e.*, this amount of heat added to the water in the instrument caused the column in the glass tube to rise 1 in. Generally, 1.5 grams of coal were burned in the calorimeter, but as a check on the work 1 gram and 2 grams were sometimes used.

The sample of coal for the calorimeter was weighed on an analytical balance, turning to $\frac{1}{10}$ mgr., in thin asbestos crucibles about $\frac{3}{4}$ in. in diameter and $\frac{3}{4}$ in. deep. These crucibles were subjected to prolonged heat over a Bunsen burner to remove any combustible matter, and cooled to room temperature in a desiccator before each determination.

Sample determinations with this calorimeter are given below:

Pure Carbon from Sugar.—1.5 grams. Scale reading 6.74 ins.

$$\text{B. T. U.} = \frac{6.74 \times 7.15}{0.00331} = 14\,560$$

Harvey Foundry Coke.—1.411 grams. Scale reading 6.15 ins.

$$\text{B. T. U.} = \frac{6.15 \times 7.15}{0.00311} = 13\,871$$

New River, Mine Run, Coal.—1.5 grams. Scale reading 7.02 ins.

$$\text{B. T. U.} = \frac{7.02 \times 7.15}{0.00331} = 15\,163$$

The Carpenter instrument, which is an improved form of the earlier Thompson Gas Calorimeter, is really a very large thermometer; the fuel being burned in the bulb, and all heat being absorbed by the fluid or lost by radiation. Combustion takes place in the presence of pure oxygen under a pressure of 9 ins. of water. The heat absorbed is accounted for by the expansion of the fluid in the vertical glass tube, while that lost by radiation is measured by the contraction of the column of fluid during an interval of time equal to the time required for the combustion of the coal sample. The expansion and contraction are measured in inches on a vertical scale placed behind the glass

*Stillman's "Engineering Chemistry," 1897, p. 120; Poole's "Calorific Power of Fuels," 1898, p. 13.

tube. The bulb of the instrument contained slightly less than 5 lbs. of water at 60° Fahr.

The precautions to be observed in the use of this calorimeter are few and easily complied with:

First.—Distilled water, absolutely free from air, should be used. This the author obtained by first gently boiling the water in a separate vessel placed over a Bunsen gas-burner, and then introducing it at high temperature into the calorimeter bulb. While cooling to nearly room temperature the bulb was so held as to cause any air which might have been trapped while filling, to seek the stuffing-box as an outlet. To prevent reabsorption of air from the atmosphere while working with the calorimeter, a column of coal oil $1\frac{1}{2}$ ins. high was run into the glass tube on top of the water.

Second.—At the beginning of each experiment, the temperature of the calorimeter and water must be sensibly higher than room temperature to avoid transfer of heat from the atmosphere to the water, and the instrument must be rigidly protected from all drafts in the room while the experiment is being conducted.

Third.—The exit orifice for the waste gases must be kept clear of all residual matters which may come over and be deposited from them. With perfect combustion there should be no residuals in the waste gases; but it is essential to good work with this instrument that the gas orifice be frequently opened with a fine brass wire, for, with continued use, in some manner this very small opening (about the size of a fine cambric needle) becomes clogged. (It is possible that the sulphuric acid in the waste gases may have a corrosive effect on the metal surrounding the exit orifice, with formation of sulphate of copper.)

Fourth.—The time required to burn the sample of coal should be noted to seconds, and the same length of time allowed for loss of heat by radiation after combustion.

Many preliminary tests with this instrument on foundry coke and pure carbon indicated an accuracy of measurement which in the comparative determination of the heat values of these coals should be all sufficient.

In Table No. 19 are given the averages of several determinations of the calorific values in B. T. U. of the different coals subjected to physical tests, together with a comparison of the ash by calorimeter, and by chemical analysis:

TABLE No. 19.—Heating Power of Coals in British Thermal Units.

Test numbers.	Kind of coal.	Heat units per pound of coal.	Ash by calorimeter.	Ash by analysis.
	Pennsylvania (Pittsburg) Coals.			
13-14-15-16.	Pittsburg, Brown, Second Pool	13 383	5.77	3.75
5-6-7-8.	" Catsburg, Third Pool	13 446	5.46	5.50
17-18.	" Cincinnati, Third Pool	13 247	4.64	4.05
9-10-11-12.	" Woods, Fourth Pool	13 316	5.66	5.60
	West Virginia Coals.			
1-31-35-49.	New River (old sample)	14 742	1.40	1.45
1-31-35-49.	" (new sample)	15 163	2.87	2.40
20-21-27.	Loup Creek (old sample)	14 137	4.17	2.15
20-21-27.	" (new sample)	14 975	1.20	0.95
22-26.	Pocahontas (old sample)	14 048	4.23	4.00
22-26.	" (new sample)	15 133	1.57	1.25
2.	Kanawha, Steam	15 163	2.33	2.10
3.	" Gas	15 088	1.33	3.75
32-33.	Acme	15 184	1.60	2.10
34.	Keystone	13 824	9.73	8.30
23-24-25.	Winifrede	14 111	2.91	2.85
28-30 37-5).	Thacker	14 526	5.88	5.00
42.	Logan Consolidated Thacker	14 245	4.13	3.85
46.	Maritime Thacker	14 126	5.20	4.60
19-29.	Campbell's Creek	14 180	3.17	2.90
36.	Monongah	14 245	3.04	3.30
51.	Eagle	14 040	3.00	2.60
52.	Cedar Grove	14 668	3.97	3.47
53.	Mt. Carbon	13 964	4.67	4.33
54.	Chesapeake	13 870	3.93	3.27
55.	Coalburg	13 781	5.26	4.93
38-39.	Belmont Splint	12 949	1.08	1.20
	Ohio Coals.			
41-43.	Luhrig, Washed	12 960	6.86	6.90
44-45.	Wellston Shaft	12 506	7.80	7.30
	Kentucky Coals.			
40-47.	Jellico	14 472	1.06	0.85
	Tennessee Coal.			
48.	Jellico	13 846	1.37	1.10

The ash, both by calorimeter and analysis, is a mean of several determinations, and, when, by the two methods of examination, the percentages of ash fail to agree, it must be charged to lack of uniformity in the samples. This variation in percentage of ash in coals from the same source is well known,* and indicates, as might be supposed, that Nature in forming the coal did not follow exact synthesis.

Some writers have recently stated the thermal values of certain coals on the net combustible, leaving out of consideration the ash and moisture, which are constant constituents of all coals as purchased for use. The propriety of this, excepting for comparison of effects of different combinations of carbon, hydrogen and oxygen in the coal,

* "The Calorific Value of Certain Coals," etc., by Lord and Haas, *Loc. cit.*, pp. 8 and 9.

may be questioned. Thus, when the constituents of the combustible are alike, or nearly alike, in two or more coals, and the B. T. U. are substantially different, such a comparison furnishes very interesting information; but when the thermal values agree very well for different coals of like composition, it is a mistake to neglect properties which are certain to be found in all coals of every nature. Neither does it furnish information of value to the consumer of coal, who is compelled to make his purchases and estimate economy with the ash and moisture included.

In connection with the heat values of the coals tested by the author, Table No. 20, comparing some of these with results obtained by other investigators on the same coals, may be of interest. In this table the data attributed to Professor Carpenter were obtained with an instrument of the same kind as that used by the author.* The data credited to Mr. Barrus were obtained from an improved form and mode of using the Thompson calorimeter.† The data credited to Professors Lord and Haas were obtained with a Mahler bomb calorimeter,‡ while those credited to Messrs. Hale and Williams were obtained with a bomb calorimeter of the Berthelot type.§

With reference to the samples of coal tested for calorific power by the author, it is proper to state that all the samples were in the office for from two weeks to eight months previous to making the calorimetric determinations, and although excluded from dust, the packages were open to free access of air, and some of the volatile matter may have been lost under the influence of room temperature. To ascertain the probable effect of keeping the coals under this condition, tests of old and new samples of New River, Loup Creek and Pocahontas coals were made, with the result, in each instance, of a higher calorific power in the new when compared with the old samples. Upon the contrary, the recent experiments of Messrs. Hale and Williams, Boston, indicate that two of the coals mentioned (New River and Pocahontas) were not impaired in heating power by exposure or weathering. With this difference, however, that exposure in the author's tests was limited to the air circulating in a room, while in Messrs. Hale and Williams' experiments the samples of coal were really exposed to the weather for eleven months.

* *Transactions*, A. S. M. E., Vol. xvi, 1895.
† *Transactions*, A. S. M. E., Vol. xiv, 1893.
‡ *Transactions*, Am. Inst. Mining Engrs., 1897.
§ *Transactions*, A. S. M. E., Vol. xx, 1899.

More than 20 years ago the author made some experiments under steam boilers with weathered and freshly mined Pittsburg coal, with the result that the freshly mined coal gave the higher amount of steam per pound of coal. The data of these early trials not having been permanently recorded, the percentage of gain of the freshly mined over weathered coal cannot now be stated, but the difference was quite material.

TABLE No. 20.—Comparison of Heat Determinations.

Kind of coal.	Heat units per pound of coal.	Mode of Determination.	Authority.
New River	14 742 (old) 15 163 (new)	Calorimeter.	Author.
"	13 858 14 455	"	G. H. Barrus.
"	14 713 15 222	Analysis.	Hale and Williams.
Pocahontas	14 048 (old) 15 138 (new)	Calorimeter.	Author.
"	15 004	"	R. C. Carpenter.
"	14 512 14 906	"	Lord and Haas.
"	13 608 14 603	"	G. H. Barrus.
"	14 975	Analysis.	Hale and Williams.
Thacker	14 526	Calorimeter.	Author.
"	13 955 14 020	"	Lord and Haas.
Pittsburg	13 247 13 316 13 389 13 446	"	Author.
"	13 237 13 977	"	Lord and Haas.
"	12 941	"	G. H. Barrus.
"	13 414 13 657 13 963 14 006	Analysis.	Hale and Williams.
Luhrig (Ohio)	12 960	Calorimeter.	Author.
Wellston (Ohio)	12 506	"	"
Hocking Valley (Ohio)	11 885 12 388	"	Lord and Haas.

Professor Johnson attempted to deduce the calorific values of the coals which came under his observation from the formulas of Despretz and Dulong* (the then prevailing expressions for the total heating power of coal), and also to relate the total heat value to that of pure carbon by the method of Berthier, which consists in reducing the lead from litharge by combustion of a known weight of coal in a crucible in the presence of an excess of the oxide of lead; the amount

* "Report to the Navy Department of the United States, on American Coals," by Professor Walter R. Johnson, 1843, p. 500.

TABLE No. 21.—Calorimeter Experiments with Harvey Coke, Gas Coke, and Pure Carbon from Recrystallized Sugar.

Sample.	Weight in grams.	B. T. U. Per pound of combustible.
Harvey Coke	1.885	13 641
"	1.411	13 860
"	1.418	14 026
"	1.413	13 987
"	0.943	13 600
"	1.885	14 357
"	0.944	13 461
Gas Coke	1.500	13 000
Pure Carbon	1.500	14 560
"	1.590	14 732

of metallic lead found, after combustion, as a button in the crucible, being weighed and related to the weight of coal consumed, and the total calorific power of the combustible obtained by proportion from the amounts of lead reduced by the given combustible, and by pure carbon.

According to Berthier, pure carbon is capable of reducing thirty-four times its weight of lead*, and by multiplying the weight of lead reduced by the calorific value of pure carbon divided by 34, the thermal value of the combustible matter in any given coal may be determined. Professor Johnson used 20 grains (about $1\frac{1}{3}$ grams) of powdered coal sample for his litharge tests. Mr. Emery has suggested that this test of Berthier for determination of the heating power of coals by reason of its simplicity and readiness of application might be revived to advantage.†

Professor Johnson made many deductions from his chemical notes to show that the percentage of fixed carbon in the combustible was a direct measure of the heating power, and that, as the ratio of carbon to volatile matter diminished, the calorific power of the coal also diminished. While this is generally true, it is now well known that the anthracite coals, high in fixed carbon and low in volatile matter, are not possessed of the highest heating powers, nor have they shown the high efficiency of the smokeless West Virginia or the Cumberland coals when submitted to physical tests.

* Johnson's Report, p. 106.

† *Loc. cit.*, p. 30.

TABLE No. 22.—Cumberland Coals.

Source.	B. T. U. per pound of combustible.	How determined.	Authority.
George's Creek.....	16 048	Bomb Calorimeter.....	Hale and Williams, 1899.
"	15 958	"	" "
"	15 799	Analysis.....	" "
" (maximum)	15 141	Thompson Calorimeter.	Barrus, 1893.
" (minimum).	14 085	"	"
N. Y. & M. Mining Co.....	13 071	Litharge Test.....	Johnson, 1843.
"Coal in Store" (Mine)....	14 089	"	"
Atkinson and Templeman,	12 954	"	"
Easby and Smith.....	13 021	"	"
John Neff	13 237	"	"
D. & S. Coal Co.....	13 395	"	"

Upon the other hand, by the litharge test, Professor Johnson found for Lackawanna anthracite coals 14 432 B. T. U. per pound of combustible, which is much nearer some of the late determinations than the results obtained by this method for the Cumberland coals. For thirteen samples of anthracite coal tested by Professor Carpenter*, he found with his calorimeter for Drifton coal 14 480 B. T. U. per pound of combustible, and from eleven samples of anthracite coal tested by Mr. Barrus,† he found for an unnamed coal 14 508 B. T. U. per pound of combustible, results quite near that obtained for Lackawanna coal by the Berthier test fifty-six years ago by Professor Johnson. Parallel coals not having been tested during the earlier and later trials, better comparisons cannot be made, but from the limited data which are at all comparable, the suggestion arises that the Berthier method may be defective when applied to coals high in volatile matter, possibly by reason of incomplete or imperfect combustion of the hydro-carbons. When applied to carbons, and coals high in carbon, it seems that the method may give reliable results.

Physical Efficiencies of the Coals.

The relation of the heat accounted for in the steam to the total heating powers of the several coals tested is given in Table No. 23. The difference between this fraction and unity, represents the losses of heat by chimney draft, evaporation of moisture in the atmosphere and coal, rendered latent in the gases of combustion, and the heat

* *Loc. cit.*, p. 17.

† *Loc. cit.*, p. 7.

TABLE No. 23.—Efficiencies Obtained From Various Coals.

Test No.	Coal sample.	Efficiency on Coal.	Efficiency on Combustible.
	Pennsylvania Coals.		
13....	Pittsburg, Brown, Second Pool, Lump	0.6500	0.6406
15....	" " " Barge Run	0.6542	0.6477
14....	" " " Mine Run	0.6081	0.6214
16....	" " " Nut and Slack	0.6091	0.6207
5....	" Catsburg, Third Pool, Lump	0.6629	0.6658
6....	" " " Barge Run	0.6315	0.6382
7....	" " " Mine Run	0.6122	0.6318
8....	" " " Nut and Slack	0.5910	0.6169
17....	" Cincinnati " Mine Run	0.6325	0.6357
18....	" " " Nut and Slack	0.6324	0.6431
9....	" Woods, Fourth Pool, Lump	0.6337	0.6557
10....	" " " Barge Run	0.6180	0.6406
11....	" " " Mine Run	0.6480	0.6731
12....	" " " Nut and Slack	0.6131	0.6406
	West Virginia Coals.		
35....	New River, Lump	0.6019	0.6056
49....	" Lump	0.6361	0.6218
1....	" Mine Run	0.6444	0.6603
31....	" Mine Run	0.6272	0.6134
27....	Loup Creek, Lump	0.6589	0.6698
20....	" Mine Run	0.5650	0.5751
21....	" Slack	0.5661	0.5805
26....	Pocahontas, Lump	0.6357	0.6627
22....	" Mine Run	0.6015	0.6333
2....	Kanawha, Steam Lump	0.5912	0.6136
3....	" Gas Lump	0.5519	0.6165
32....	Acme, Mine Run	0.6086	0.6311
33....	" Nut and Slack	0.5768	0.6156
34....	Keystone, Mine Run	0.6162	0.6058
23....	Winifrede, Lump	0.6306	0.6514
24....	" Mine Run	0.5881	0.5989
25....	" Nut and Slack	0.5710	0.6300
37....	Thacker, Lump	0.6294	0.6217
28....	" Mine Run	0.6246	0.6293
50....	" Mine Run	0.6332	0.6291
30....	" Nut and Slack	0.6012	0.6288
42....	Logan Consolidated Thacker, Mine Run	0.6210	0.6327
46....	Maritime Thacker, Mine Run	0.6335	0.6230
29....	Campbell's Creek, Mine Run	0.6079	0.6390
19....	" Nut and Slack	0.5381	0.5676
36....	Monongah, Mine Run	0.6062	0.6162
51....	Eagle, Mine Run	0.6223	0.6522
52....	Cedar Grove, Mine Run	0.6168	0.6164
53....	Mt. Carbon, Mine Run	0.6171	0.6267
54....	Chesapeake, Mine Run	0.6370	0.6396
55....	Coalburg, Mine Run	0.6128	0.6187
38....	Belmont Splint, Mine Run	0.6513	0.6909
39....	" Nut and Slack	0.6081	0.6754
	Ohio Coals.		
41....	Luhrig, Washed Egg	0.5829	0.5655
43....	" " Slack	0.5423	0.5159
44....	Wellston Shaft, Mine Run	0.6230	0.5695
45....	" Nut and Slack	0.5847	0.5450
	Kentucky Coals.		
47....	Jellico, Mine Run	0.6084	0.6123
40....	" Nut and Slack	0.5192	0.5645
	Tennessee Coal.		
48....	Jellico, Mine Run	0.6083	0.6081

losses by radiation and conduction. Notes sufficient to divide up the gross loss into the detailed losses were not taken, nor with the object which these tests had in view was it necessary, because the boiler, furnace and chimney conditions were alike for all coals. Exceptions should be made of the moisture in the atmosphere, for which neither coal nor boilers should be held responsible; but this, in the author's previous experience, has never exceeded $1\frac{1}{2}\%$ of the total heat of combustion.

Proximate Analyses of Coals.

The notes on the proximate analyses of these coals (Table No. 24) are means of three to five determinations for each sample. In these tests the sample of powdered coal was carefully dried at 105° Cent. for one hour, and then re-weighed for loss by moisture. The first few samples were then returned to the gas oven for fifteen or twenty minutes to ascertain if the additional exposure to the dry heat produced further loss of weight due to moisture. The early tests demonstrated that one hour was sufficient to expel the moisture from the samples, and all the after determinations were made upon this time of exposure to dry heat. The sample in the covered porcelain crucible was then exposed to the naked flame of a Bunsen burner until it ceased to give off gas at the edge of the cover. The crucible, without cooling, was then held over a blast lamp at white heat for three minutes, to expel the small amount of volatile matter which remained after the gas flame was extinguished. The crucible and contents were then cooled in a desiccator and weighed for the loss of weight of sample by volatile matter. The fixed carbon remaining in the crucible was then burned to constant weight in a current of oxygen over a Bunsen burner, after which operation the crucible was again cooled in the desiccator and weighed for loss due to the fixed carbon. The difference between the last weighing and weight of crucible represented the ash or non-combustible in the sample. No determinations for sulphur in the combustible or phosphorus in the ash were made, partly because of lack of significance of these elements in steam coals, and partly for lack of time, the city officials desiring the *résumé* of the work at an early date to use in connection with the annual contract for city coals.

The samples for proximate analysis and for the calorimetric deter-

minations were obtained by the customary method of quartering from several pounds to the small weights finally used. It was believed that all the samples excepting the fresh samples of New River (1), Pocahontas (26), Loup Creek (27), Eagle (51), Cedar Grove (52), Mt. Carbon (53), Chesapeake (54) and Coalburg (55), had lost in volatile matter during the interval of time which elapsed between the collection of the sample from the lot of test coal fired under the boilers, and the tests for moisture and combustible matter, and for heating power; but careful consideration of the proximate analyses of the old and new samples of New River, Loup Creek and Pocahontas coals, suggest that this may not generally have been true.

TABLE No. 24.—Proximate Analyses of Coals Tested.

Sample of coal.	Specific gravity.	Moisture.	Volatile matter.	Fixed carbon.	Ash.
Pennsylvania Coals.					
Pittsburg, Brown, Second Pool	1.317	1.17	26.93	68.15	3.75
" Catsburg, Third Pool	1.291	0.76	33.04	60.70	5.50
" Cincinnati, Third Pool	1.295	0.98	33.37	61.60	4.05
" Woods, Fourth Pool	1.292	0.65	31.95	61.80	5.60
West Virginia Coals.					
New River (old sample)	1.270	0.50	17.85	80.20	1.45
" (new sample)	1.270	0.90	18.60	78.10	2.40
Loup Creek (old sample)	1.275	0.55	19.25	78.05	2.15
" (new sample)	1.275	1.15	19.15	78.75	0.95
Pocahontas (old sample)	1.277	0.45	14.95	80.60	4.00
" (new sample)	1.277	0.52	18.80	79.43	1.25
Kanawha, Steam	1.277	0.80	31.28	65.82	2.10
" Gas	1.261	0.95	37.35	57.95	3.75
Acme	1.273	1.00	31.50	65.40	2.10
Keystone	1.316	0.78	28.92	62.00	8.30
Winifrede	1.297	1.05	32.40	63.70	2.85
Thacker	1.324	0.73	31.32	62.95	5.00
Logan Consolidated Thacker	1.309	0.70	32.60	62.85	3.85
Maritime Thacker	1.301	0.68	31.67	63.05	4.60
Campbell's Creek	1.278	0.95	38.00	58.15	2.90
Monongah	1.282	1.05	33.15	62.50	3.30
Eagle	1.268	1.00	30.53	65.87	2.60
Cedar Grove	1.308	0.92	35.35	60.26	3.47
Mt. Carbon	1.266	0.73	31.87	63.07	4.33
Chesapeake	1.274	1.15	34.28	61.30	3.27
Coalburg	1.289	0.90	38.17	56.00	4.93
Belmont Splint	1.269	1.28	34.22	63.30	1.20
Ohio Coals.					
Luhrig, Washed	1.306	2.87	33.47	56.76	6.9
Wellston, Shaft	1.293	3.38	34.07	55.25	7.3
Kentucky Coal.					
Jellico	1.273	1.68	36.62	60.85	0.85
Tennessee Coal.					
Jellico	1.275	1.80	36.10	61.00	1.10

(The above determinations were made by Mr. Henry C. Hill.)

Kent's Estimate of Total Heat of Coals from Proximate Analysis.

Mr. William Kent has given a table of the proximate heating values of coals deduced from Mahler's experimental data.* In this table he attempts to show, within limits of error of 3%, the B. T. U. of coals, having previously determined percentages of fixed carbon of the net combustible. The figures in this table have been plotted as co-ordinates on the diagram (Fig. 7) and connected by a smooth curve. Upon the diagram the heat values obtained by the author with the Carpenter calorimeter, together with the results of tests by Professors Carpenter, Lord and Haas, and Hale and Williams, have also been plotted for comparison with Mr. Kent's curve. An examination of this diagram indicates a wide difference between the heating values of certain coals as given by the curve and from calorimetric tests, and this doubtless is due to the variable nature of the volatile matter.

From any series of experiments on bituminous coals like those by Mahler, curves can be obtained which will approximately fit the present conditions, but not be generally applicable to another series of tests of coals from other localities. By reference to Table No. 25 it will be seen that four of the best coals calorimetrically tested, viz: Thacker, Acme, Kanawha Steam and Kanawha Gas coals, having heating powers of 15 554, 15 589, 15 653 and 15 440 B. T. U. per pound of net combustible, contained scarcely two-thirds of fixed carbon, and by the Kent method should have rated in heating power considerably lower than the smokeless coals; when, in fact, by the calorimetric method these coals were not inferior to the coals giving the highest heating powers and evaporative results.

Referring again to the table it will be noticed, that as a rule, the actual heating power of the net combustible† was considerably under the value which would be assigned to it by the Kent curve. It is scarcely probable that the heats obtained by the author were too low, because combustion must have been perfect in the presence of pure oxygen under pressure, and as shown by the absence of smoke and soot in the combustion chamber, and at the orifice of exit for the waste gases; and was complete as shown by the fair agreement of the ash by calorimetric method and proximate analysis.

* Kent's "Mechanical Engineers' Pocketbook," First Edition, p. 634.

† Coal—(ash + moisture).

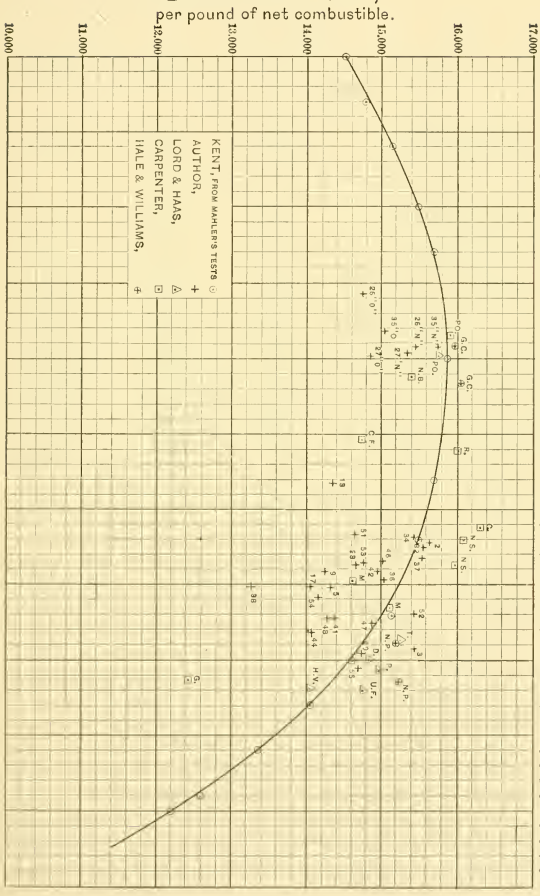

British Thermal Units,
per pound of net combustible.
17,000
16,000
15,000
14,000
13,000
12,000
11,000
10,000
100 95 90 85 80 75 70 65 60 55 50 45
Percentage of Fixed Carbon in the Net Combustible.
KENT, FROM MAHLER'S TESTS
AUTHOR,
LORD & HAAS,
CARPENTER,
HALE & WILLIAMS,

FIG. 7.

TABLE No. 25.—Heating Values of Net Combustible.

Coal.	Percentage of fixed carbon in net combustible.	B. T. U. Per Pound of Net Combustible.	
		By calorimeter.	By Kent's curve.
Pennsylvania Coals.			
Pittsburg, Brown, Second Pool	71.70	14 381	15 670
" Catsburg, Third Pool	64.75	14 317	15 260
" Cincinnati, " "	64.86	14 036	15 270
" Woods, Fourth "	65.93	14 213	15 350
West Virginia Coals.			
New River (old sample)	81.80	15 028	15 815
" (new sample)	80.77	15 757	15 840
Loup Creek (old sample)	80.22	14 837	15 850
" (new sample)	80.44	15 336	15 845
Pocahontas (old sample)	84.35	14 738	15 770
" (new sample)	80.86	15 456	15 840
Kanawha Steam	67.79	15 653	15 470
" Gas	60.81	15 440	14 780
Acme	67.49	15 589	15 460
Keystone	68.19	15 448	15 490
Winifrede	66.29	14 693	15 375
Thacker	66.78	15 554	15 490
Logan Consolidated Thacker	65.85	14 968	15 345
Maritime Thacker	66.56	15 008	15 380
Campbell's Creek	60.48	14 789	14 690
Monongah	65.34	15 022	15 300
Eagle	68.32	14 625	15 500
Cedar Grove	63.03	15 482	15 120
Mount Carbon	66.43	14 761	15 380
Chesapeake	64.13	14 086	15 210
Coalburg	59.47	14 686	14 510
Belmont Splint	64.91	13 262	15 275
Ohio Coals.			
Luhrig, Washed	62.92	14 357	15 105
Wellston Shaft	61.86	14 080	14 960
Kentucky Coal.			
Jellico	62.43	14 880	15 030
Tennessee Coal.			
Jellico	62.82	14 299	15 100

One serious objection to the general application of Mr. Kent's curve is the variable nature of the volatile matter in bituminous coals, which may have an effect on the calorific power Thus, with constant fixed carbon, when the hydrogen is relatively high and the oxygen relatively low, the heating power of the volatile matter will be high, and when the hydrogen is relatively low and the oxygen relatively high, the heating power of the volatile matter will be low. (Varying amounts of sulphur in the coal will have the effect of slightly modifying this.)

Table No. 26, from analyses by Professors Lord and Haas,* will show the influence of varying relative amounts of hydrogen and oxygen on the heating power of coals:

TABLE No. 26.—Relation of Hydrogen, Oxygen and Heating Power of Coals.

Test No.	Coal.	Percentages.				Heating Power.	
		Ash.	Fixed Carbon.	Hydrogen.	Oxygen.	By calorimeter. Calories.†	By analysis. Calories.†
24	Freeport.....	9.17	73.15	4.98	7.41	7 464	7 393
30	"	9.10	71.40	4.62	10.68	7 113	6 970
16	Pittsburg.....	7.95	76.56	5.22	7.00	7 630	7 719
34	"	9.05	73.91	5.15	8.89	7 354	7 404
32	Darlington.....	8.70	72.78	4.93	10.57	7 245	7 173
33	"	8.80	72.82	5.25	8.55	7 304	7 395
6	Hocking Valley.	5.83	69.42	5.35	16.27	6 882	6 790
8	" " .	9.67	66.59	5.16	15.57	6 494	6 520
11	Thacker.....	6.50	78.90	5.14	6.88	7 768	7 876
14	"	6.05	78.40	5.19	7.56	7 867	7 831
3	Pocahontas.....	5.63	85.46	4.25	3.24	8 185	8 246
5	"	4.80	85.40	4.39	3.94	8 280	8 258

† Pound calories × 1.8 = B. T. U.

Considering the Freeport, Pittsburg and Darlington coals, the relation of hydrogen and oxygen varies sufficiently to overcome the modifying influence of the differences in percentages of fixed carbon. With the Hocking Valley coal the relation of the hydrogen and oxygen is fairly constant, but the combustible varies in heating power by reason of the variable proportion of fixed carbon, while with the Thacker and Pocahontas coals all conditions of the combustible are fairly constant, and the heating powers likewise are constant.

The data in Table No. 27, from ten samples of bituminous coals, are taken from Professor Carpenter's paper, descriptive of his form of coal calorimeter,‡ and compared on the heating power of the net combustible with the heats obtained for the given percentages of free carbon by Mr. Kent's curve.

* *Loc. cit.*

‡ *Loc. cit.*

TABLE No. 27.

Source of coal.	Percentage of free carbon.	B. T. U. Per Pound of Net Combustible.	
		By calorimeter.	By Kent's curve.
Nova Scotia	67.93	16 093	15 475
" "	66.27	15 964	15 340
Pennsylvania	65.19	14 601	15 290
"	73.93	16 000	15 750
"	68.78	16 315	15 510
"	63.47	15 106	15 160
"	74.73	14 753	15 780
"	78.87	15 396	15 835
West Virginia	81.47	15 872	15 820
Illinois	58.65	12 435	14 400

Specific Gravity of the Coals.

The specific gravity given in the table of analyses of each kind of coal—a mean of five or six determinations—will be found of value in estimating the stowage for marine, railway and factory uses where the coal storage space is limited. Thus a coal of given specific gravity and heating power will occupy less bulk for a given steam effect than will another coal lower in specific gravity and of equal heating power. The relative values where stowage is a consideration will be approximately represented by the products of the heating powers and specific gravities of the several coals, and to illustrate the effect of this, the following comparisons of a few coals have been made:

TABLE No. 28.

Coal sample.	Product, heating power (B. T. U.), and specific gravity.	Relative stowage value.
Pittsburg (average)	17 339	89.70
New River	19 257	99.63
Loup Creek	19 063	98.78
Pocahontas	19 225	100.00
Acme	19 329	100.00
Thacker	19 232	99.50
Eagle	17 803	92.10
Cedar Grove	19 186	99.26
Luhrig, Washed	16 926	87.57
Wellston Shaft	16 170	83.66

Considering that the New River, Loup Creek, Pocahontas and Thacker coals, in addition to their other good qualities, are all so-

called smokeless coals, the importance of these for naval and railway purposes is quite apparent.

It should be matter for congratulation by the people of the United States residing east of the Mississippi River, that Nature has been so bountiful in locating such vast tracts of high-grade steam coals as are found in the States of Pennsylvania, West Virginia and Kentucky, especially as these coals are easily mined and cheaply transported to the seaboard, or to points of consumption inland. Some of these coals, like Pocahontas, New River, and Acme, are not surpassed for steam use by any known coals in the world, and when boilers and furnaces have been devised to obtain the highest practical efficiencies, then these coals will be made to contribute in a higher degree to the commercial prosperity of the country.

Reference to the physical efficiencies of these coals (Table No. 23), indicates that none of the coals was giving very high results, and some, undoubtedly, were worked at a disadvantage, although the evidences of this were not apparent at the time; but high efficiency of the low-grade coals cannot always be attained except in furnaces especially adapted for them. All the losses of heat by chimney draft, etc., should be limited to 30% of the ultimate heat value of the coal; and in designing boilers and furnaces, due regard should be had for the work to be performed and the coal to be used. But variations in any large degree in the amount of work, and the kind and grade of coal burned, will in most instances be attended with reduced efficiency of the fuel.

DISCUSSION.

Mr. Kent. Mr. William Kent.—This paper is a valuable contribution to the knowledge of the heating value of some of the most important American coals, and Mr. Hill is to be congratulated on having presented one of the most important papers on this subject which has yet appeared.

The author has had an opportunity which few engineers have ever obtained of making a systematic series of tests of coals mined over a wide extent of territory, with conditions under his control, and with facilities for making both chemical and calorimetric tests.

All the results which the opportunity offered have not been obtained by the author, nor has he drawn all the conclusions which might have been drawn, and the speaker will have to find fault with some details of his methods and with some of his figures.

The map, Fig. 2, showing the location of the coal fields, is especially to be commended. The chief factor which determines the heating value of any of the coals mined east of the Mississippi is the geographical location of the mine. No satisfactory study of American coals can be made without a map showing the location of the mines.

In regard to the methods of testing, the author was fortunate in having an internally fired boiler which could be driven at a uniform rate and thus keep the loss by radiation practically constant. He was unfortunate, however, in having the rate of evaporation between 5 and 6 lbs. per square foot of heating surface per hour, so high as to be far beyond the point of maximum economical performance. He was also unfortunate in not having a furnace and combustion chamber surrounded by fire brick, which would have been of great assistance in burning completely those coals which had over over 30% of volatile matter.

The author says:

"The results given in the tables are in no case supposed to indicate the best possibilities with these coals. * * * The author regards the results obtained by this method of handling the coal as more satisfactory to the buyers and users of coal than those which might have been obtained by expert firemen."

The results are certainly below the best possibilities of the coals. They could not be anything else, with the boiler driven beyond its economical rating, with a furnace not adapted for burning the volatile gases, and with only the ordinary firemen. This may be satisfactory to the buyer and user of the coal, but it should not be satisfactory to the seller of the coal nor to an engineer. The object of the tests appears to have been to determine the relative value of the coals. If each coal had been tested in such a manner as to develop its best possible results, with a clean boiler, with expert firemen and with

a boiler not over driven, the results then could be taken as expressing the true relative value of the different coals. But when conditions are unfavorable, as they were in these tests, there is no way of determining that they are not more unfavorable for one coal than for another. Unfavorable conditions are variable conditions, and they should be avoided in testing coals, boilers and engines, just as much as in racing horses. Mr. Kent.

An example of the effect of what was probably variable firing is seen in the figures for Pittsburg coal, for the item "pounds of steam from and at 212° per pound of combustible" in Tables Nos. 5 and 6:

Name.	Lump.	Barge Run.	Mine Run.	Nut and Slack.
Brown	9.54	9.64	9.25	9.24
Catsburg	9.87	9.46	9.36	9.14
Cincinnati	....	....	9.24	9.34
Woods	9.65	9.43	9.90	9.43

There can be no doubt that these coals were practically of the same quality, and it might be expected that these 14 tests would give some knowledge of the relative value of different sizes. But the highest figures are given by the Barge Run for the Brown coal, by the Lump for the Catsburg, by the Nut and Slack for the Cincinnati, and by the Mine Run for the Woods. The Brown coal is about 2% better than the Woods, if judged from the Barge Run tests; but 7% poorer than the Woods, if judged by the Mine Run tests.

The author has something to say concerning a curve which the speaker plotted from the results of Mahler's experiments in France in 1892. *

The speaker will not undertake at present a defence of this curve, but will only say that the results presented in the paper have not caused him to modify in any degree the latest statements he has made in relation to it, giving the limitations of its usefulness.

The results given in the paper will now be taken up and a few conclusions drawn therefrom.

The data give three different ways of estimating the probable heating value of the several coals: (1) The determinations by the Carpenter calorimeter, (2) the approximations by the use of the speaker's curve, and (3) the results of the boiler tests. Each of these three methods is subject to error.

The Carpenter calorimeter is by no means recognized as being as accurate an instrument as the Mahler calorimeter. It is greatly to be regretted that Mr. Hill did not have his results checked by Professor Lord, at Columbus, with his Mahler calorimeter. The standardizing and the manipulation of the Carpenter calorimeter is a matter of great

* "Mineral Industry," Vol. I, p. 97, and revised with the addition of the results of Lord and Haas, in 1898; *Transactions*, A. I. M. E., Vol. xxvii. p. 955; and *Transactions*, A. S. M. E., Vol. xx.

Mr. Kent. delicacy, and any work done by it, to have scientific value, should be done by an expert.

The use of the speaker's curve depends on accuracy in the proximate analysis of the coal, which is not easy to obtain, as may be found by sending duplicate samples of coal to three or more chemists for analysis. The speaker does not claim for his curve a greater approximation to accuracy than 3%, even when the analysis is accurate; and it is liable to a somewhat greater error than this in coals which are high in volatile matter, such as the Pittsburg and Thacker coals.

The best boiler tests are subject to an error of about 3%, and, as already shown, these particular boiler tests show some evidence of having even larger errors, or, more properly, larger variations from the results which were possible under the given conditions.

There is a strong probability that the efficiency of the boiler, barring unavoidable variations due to irregularity of firing and the ordinary errors of a test, is approximately uniform for all the coals of a given class, and if we assume a certain efficiency of boiler for these tests, we will arrive thereby at an approximation to the heating value of the coal. There can be no such error in this method as there is apparently in the calorimetric results obtained by Mr. Hill. For instance, in Table No. 23 the efficiency of the boiler when testing Loup Creek, Mine Run coal was only 57.51%, while it was 69.09% with the Belmont Splint, Mine Run, or over 20% higher. This is only partly explained by the poor result of the boiler test with the Loup Creek coal, and it must be largely due to an incorrect calorimetric result with the Belmont coal.

It may be assumed, for the purpose of this study, that the three different estimates of the heating value of these coals are of equal credibility, and that a somewhat near approximation to their true heating value can be obtained by averaging the three estimates. To obtain a convenient approximation to the heating value of the coal from the boiler tests, the speaker has multiplied the figures representing the evaporation per pound of combustible by 1 500 for the Pocahontas, New River and Loup Creek coals, and by 1 600 for all the other coals. This is equivalent to assuming a boiler efficiency of 64.38% for the three coals first named, and 60.36% for the other coals*, which is a fair assumption, since the three coals are of a distinct class, semi-bituminous, and always, in internally fired boilers, give higher efficiencies than the bituminous coals.

From the several results the speaker has selected the best obtained from each coal, whether Lump, Barge Run, Mine Run, or Nut and Slack, thus giving each coal the advantage of the best showing it made. This

*10 lbs. evaporation $\times$ 1 500 = 15 000 B. T. U.
$10 \times 965.7 \div 15\,000 = 64.38\,\%$.
10 lbs. evaporation $\times$ 1 600 = 16 000 B. T. U.
$10 \times 965.7 \div 16\,000 = 60.36\%$.

seems to be only fair to each coal, and it does not seem fair to depre- Mr. Kent.
ciate the value of Loup Creek, which gave 10.63 lbs. evaporation in Lump coal by crediting it with the extremely low figure of 9.13 for the Mine Run coal, which was no doubt due to improper firing.

Making the calculations in this way, the results given in Table No. 29 have been obtained.

TABLE No. 29.—B. T. U. per Pound of Combustible.

	1	2	3	4	5	6	7
	By Calorimeter.	By Kent's curve.	By boiler test.*	Average.	Average greater or less than		
					1	2	3
Pittsburg, Brown, 2d Pool	14 381	15 670	15 427	15 159	778	—511	—268
" Catsburg, 3d Pool	14 317	15 260	15 784	15 120	803	—140	—664
" Cincinnati, 3d Pool	14 036	15 270	14 950	15 073†	1 037	—197	123
" Woods, 4th	14 213	15 350	15 847	15 137	924	—213	—710
New River	15 757	15 840	16 167	15 921	174	—81	—246
Loup Creek	15 396	15 845	15 950	15 710	374	—135	—240
Pocahontas	15 456	15 840	15 905	15 734	278	—106	—171
Kanawha Steam	15 653	15 470	15 909	15 677	24	207	—232
" Gas	15 440	14 780	15 766	15 329	—111	549	—437
Acme	15 589	15 460	16 296	15 782	193	322	—514
Keystone	15 448	15 490	15 501	15 480	32	— 10	— 21
Winifrede	14 693	15 375	15 853	15 307	614	— 68	—546
Thacker	15 554	15 400	16 205	15 720	166	320	—485
Logan, Cons. Thacker	14 968	15 345	15 686	15 333	365	— 12	—353
Maritime, Thacker	15 008	15 380	15 487	15 292	284	— 88	—195
Campbell's Creek	14 789	14 690	15 455	14 978	189	288	—477
Monongah	15 022	15 300	15 331	15 218	196	— 82	—113
Eagle	14 625	15 500	15 708	15 278	653	—192	—430
Cedar Grove	15 422	15 125	15 746	15 429	7	304	—317
Mt. Carbon	14 761	15 380	15 322	15 154	393	—226	—168
Chesapeake	14 086	15 210	14 921	14 739	653	—471	—182
Coalburg	14 686	14 510	15 050	14 749	63	239	—301
Belmont Splint	13 262	15 275	15 076	14 538	1 276	—737	—538

*Calculated by multiplying the pounds of steam from and at 212° by 1 500 for the New River, Loup Creek and Pocahontas coals, and by 1 600 for all the other coals. This is equivalent to assuming a boiler efficiency of 64.5% for the three coals named and 60.4% for the other coals.

†Average of columns 2 and 3 only, the figures in column 1 being considered incredible.

It will be noticed that the average figures in column 4 show a much higher degree of uniformity than the figures in columns 1 and 3, and a somewhat higher degree of uniformity than the figures in column 2. The last three columns show the differences of columns 1, 2 and 3 from the average. Comparing these figures of differences the following results are found: Of the twenty-three coals the maximum difference between the average and the three individual results for each coal occurs 15 times in column 5, representing the variation of the Carpenter calorimeter results from the average; once in column 6, representing the variation of the results obtained by the Kent curve from the average; and 7 times in column 7, representing the variation

Mr. Kent. between the boiler test results and the average. The minimum difference, or the nearest approximation to the average value, occurs 7 times in column 5, 11 times in column 6, and 5 times in column 7. The average difference is 418 in column 5, 239 in column 6, and 339 in column 7.

Thus, by three different methods of comparison the Kent curve shows a closer approximation than either the calorimeter or the boiler test to the average heating values obtained from the three different methods of estimation.

The speaker appends the following remarks, as showing the conclusions he has reached from earlier studies in relation to the heating values of semi-bituminous and bituminous coals mined in different districts, which conclusions are confirmed by this study of Mr. Hill's paper.

Draw a line northeasterly from the Pocahontas field, shown at No. 6 on the map, Fig. 2, on the boundary line between Virginia and West Virginia, through Maryland a little west of Cumberland and through Somerset, Cambria and Clearfield counties in Pennsylvania. This line, 250 miles in length, will pass through the coal fields containing the best steam coals mined in the United States. A remarkable feature of these coals is their uniformity in chemical composition, their combustible portion varying only within 2 or 3% from 80% fixed carbon and 20% volatile matter. The heating power is likewise remarkably uniform, averaging 15 750 B. T. U. per pound of combustible, with a probable maximum variation of not over 2% on each side of this figure. The ash and moisture of all the coals along this line are also very low, the ash varying no doubt with the care used in preparing the coal for market at the individual mines. These coals are properly classed as semi-bituminous.

Traveling northwestwardly, at right angles to and from any point on this line, it is found that the coal gradually increases in volatile matter, and decreases in heating value. In the coals mined on the line the volatile matter is only about 20% of the total weight of the combustible, but it is all nearly of the composition of marsh gas, $C H_4$, having but little oxygen; and every pound of it is worth nearly $1\frac{1}{2}$ lbs. of carbon. Further to the northwest, however, while the proportion of volatile matter increases, it becomes of poorer quality, being higher in oxygen. In some parts of Ohio the volatile matter is worth no more per pound than carbon, and in most Illinois coals it is worth less.

Referring to the map, Fig. 2, it is found that, with the exception of the Jackson, Ohio, and Jellico, Kentucky and Tennessee coals, of which the author had only a few samples, nearly all the coals tested by him were mined not very far distant from the northeast and southwest line described. The Pocahontas, New River and Loup Creek

coals are on the line, and their heating value averages approximately 15 800 B. T. U. per pound of combustible. The Pittsburg, Monongah, Winefrede and Kanawha, and Thacker fields lie at some distance to the northwest of the line. The Pittsburg coals are the farthest away from the line, and their probable average heating value is 15 100 B. T. U. All the other coals, of the fields previously named, from Fayette, Kanawha and Mingo counties, West Virginia, from sixteen different mines, scattered along a northeast and southwest line over 150 miles in length, have a heating value the entire range of which, (omitting the Belmont Splint coal, the apparent average heating value of which is no doubt too low), is only between the figures 14 739 and 15 782 B. T. U. per pound of combustible. The average of these figures is 15 260 B. T. U., and the range on either side of this average is only 3.4 per cent. This result is a confirmation of the conclusion reached by Professors Lord and Haas in their paper in the *Transactions* of the American Institute of Mining Engineers, to the effect that the coals mined in a given seam, over a considerable area of territory, have a substantially uniform heating value per pound of the combustible portion. Mr. Kent.

Mr. William H. Bryan (by letter).—This paper represents such an amount of experimental research, and the results have been compiled with such great labor and care, that considerable study is necessary to fit one to discuss it intelligently. This, taken with the author's high standing as an engineer, would, for the most part, tend to disarm criticism. The paper is unquestionably a most valuable addition to the data on the value of steam coals in everyday steam-boiler service. Mr. Bryan.

The following points of criticism, however, would probably occur to the average engineer familiar with boiler trials:

First.—The starting and stopping of the tests do not seem to have been done by either the "standard" or the "alternate" method, recommended by the code of the American Society of Mechanical Engineers, which for many years has represented the best American practice. It is to be regretted that one or the other plan was not followed rigidly. The author states that the time of beginning and closing a test was when the coal charge on the grates was no longer able to maintain working steam pressure. This seems rather indefinite, and more likely to introduce error than either the "standard" or "alternate" method referred to. The writer's preference is for the "alternate" method, which he thinks would have given excellent results in the present case; and he doubts whether Mr. Hill's method insures that the quantity and condition of the fuel on the grate are the same at the beginning and end of the test. The duration is, in all except six tests, exactly 16 hours, which leads to the suspicion that exactness as to the conditions might possibly have been sacrificed to uniformity of time. The duration of the tests, however, was suffi-

Mr. Bryan. ciently long to eliminate largely any errors which might have crept in, in starting and stopping.

Second.—The use of meters for measuring the water seems unfortunate, in view of their known unreliability. The system of calibration was, of course, such as to reduce the possibility of error to a minimum, but the use of the ordinary weighing or measuring tanks would not have greatly increased the complication, and would have eliminated doubt.

Third.—Table No. 4 is a comparison of the boilers when clean and when dirty, and shows a fuel saving of $2\frac{1}{4}\%$ in favor of the clean boilers. The cleaning, however, seems to have been entirely internal, and no data are given as to the effect of soot on the external surfaces. The writer has found this to be a much more serious matter than scale, cutting down capacity and efficiency very rapidly. Many of the coals tested were very smoky. It is to be regretted that Mr. Hill did not at the same time make some record of the smokelessness of each of these coals, as they no doubt differed greatly in this characteristic. The question of smoke abatement is a live one in Cincinnati just now.

Fourth.—Mr. Hill's method of determining the accidental moisture in coal by drying a sample on the man-head is a good one. Great care must be taken, however, to insure that a representative average sample is secured. A still better plan is to air-dry, in a warm room, the entire amount of coal to be burned, whenever it is possible to do so.

The above criticisms, it will be noticed, are on details of minor importance, and do not affect the value of the paper as a whole. In the writer's judgment the results are not only strictly comparative, but their absolute value is great, even though the author's methods may not in every detail meet the requirements of the highest technical refinement.

Mr. Hill has done well to call attention to the effect of external conditions upon the quality of the steam. When a boiler is crowded beyond its rated capacity, or when its rate of work is changed suddenly and materially, or where the condition of the fires is altered suddenly, it is natural for the effect to be felt in increased moisture in the steam. It is also true, however, that a well-designed boiler will keep these disturbances at a minimum.

The surprising fact that increased stack temperatures are sometimes accompanied by higher boiler efficiencies has been noticed by others, and is probably due to the fact that combustion is more complete when the fire-box temperatures are high, and that the heat transmission to the water is better on account of the greater differences of temperature. It is also probable that a much smaller excess of free air is admitted to the grates at such times.

Mr. Carpenter. Mr. R. C. Carpenter (by letter).—In connection with the laboratory work at Sibley College, Cornell University, various forms of coal

Mr. Carpenter.

calorimeters have been used, and the results obtained compared with different calorimeters, and also in a number of cases the determinations have been compared with the computations made from a complete chemical analysis.

As a general conclusion it is probably possible to get accurate results with all the commercial forms which admit of standardization. In fact, it is quite possible to get identical results, by proper management and correct manipulation, with any of these instruments. The question of inaccuracy cannot be raised so much with reference to the instrument used as to the personal habits and methods of manipulation, as it is found that nearly all the errors in calorimetric determinations which are made by students are due to personal errors of observation. The errors most likely to be made are those pertaining to the weighings and to the determination of moisture which may occur in the samples of coal during the manipulations.

The various errors which are made in calorimetric determinations, other than those pertaining to the instrument, may be summed up as follows:

(1) Errors in sampling.
(2) Errors in weighing the sample or the residue of ash, etc.
(3) Errors caused by change of moisture in the sample during the various manipulations.
(4) Errors in weights of water in the calorimeter.
(5) Errors in rise of temperature of water.
(6) Instrumental errors.

The writer will consider these various errors in the above order.

(1) Regarding the errors in sampling, the writer's experience has shown that if the directions for obtaining the sample are properly followed, little or no error should result from this cause. The practical proof of this assertion is the uniformity of results which have been obtained in the laboratory work referred to with a great number of samples of coal, taken not only from the same carload, but from different carloads from the same mine. In fact, these investigations rather serve to show a decided uniformity in the value of the combustible matter of the coal taken from an entire region. The only prominent exception to this is the coal from the Clearfield District, which seems to vary widely in character. The coals from most of the other districts, while differing greatly from each other, agree very closely when from the same mine. Thus there is reason to expect that a coal from the Lehigh anthracite region will show very closely 14 500 B. T. U. per pound of combustible; the coals from the districts further west and which are classified as semi-anthracites, usually give a heating value of nearly 15 000 B. T. U. per pound of combustible. This value is also very closely realized by the bituminous coals in the Pittsburg district. The coals found in about the same latitude

Mr. Carpenter. as Pittsburg show a gradual falling off in heating power as the source of supply is removed westward and an increase in amount of moisture. The Hocking Valley coals have a heating value of about 14 500 B. T. U. per pound of combustible, while those still further west show a considerable falling off in value; some of the Illinois coals showing as low as 12 000 B. T. U. per pound of combustible and 8 000 or 10 000 B. T. U. per pound of coal. This regularity in the quality of coal from the same district neutralizes very greatly the effect of errors of sampling.

(2) The sample usually selected for calorimetric determination has a weight of about 1 gramme, approximately $\frac{1}{250}$ lb. It is not feasible to burn very much larger samples of coal because of the expense of the necessary oxygen, nor is it necessary, provided the weighing be accurately done on a chemical balance. The difficulties of securing perfect combustion would also increase were the sample made much larger. In order to do accurate weighing, a person must understand the use of a chemical balance. This is not a difficult machine to master, but the writer finds from experience that engineers are so accustomed to work with large quantities that more errors are made in its use than in performing any other operation connected with the calorimetric determination, except the temperature readings. There is, of course, no excuse for this condition of affairs, except that engineers are not trained to make fine weighings. Any error in weighing affects the results in a direct proportion.

(3) The samples of coal should be shipped in a hermetically sealed jar, so as to retain the original amount of moisture. Coal gives up or takes up moisture very readily and very rapidly, as it may be drier or damper than the air of the room. This change is so rapid that, if the coal as originally received is burned in the calorimeter, considerable error may result from a change of moisture contents sufficient to affect seriously the results, between the time of taking the first weights and of getting the coal into the calorimeter. This error can be obviated, in large part, by making all determinations on very dry coal. The writer once presented a paper before the American Society of Mechanical Engineers describing the Hygrometric Qualities of Coal.

(4) Errors of weight of water in the calorimeter affect the results directly, since, in most cases, the heating value is determined from the product of rise of temperature and weight of water; the water used is ordinarily from 300 to 800 times that of the fuel burned.

(5) Errors in rise of temperature may be due to lack of uniform temperature in the water and to errors in thermometer readings. On account of the large amount of water in relation to the fuel burned, the temperature change is very small, and great care must be taken in temperature determinations. These are fully as likely to occur as errors in weighing. They are minimized by using a very accurate thermometer and taking the readings with great care.

Mr. Carpenter.

(6) The errors due to the instrument itself and inherent in it can always be eliminated by a process of standardization. The best method of standardizing is, doubtless, the burning of charcoal made from sugar, and this is recommended by the German chemists as the most reliable method of determining instrumental constants.

It appears from the paper that Mr. Hill used for his work the writer's calorimeter; unfortunately, his determinations have been made on coals which the writer has not previously tested, and consequently he is unable to give any results which could be used to verify those in the paper. The results seem, however, from a knowledge of coals of adjacent regions, to be reasonable, and what would be expected.

Furthermore, judging from the method of use of the instrument, and of the care and accuracy with which the experiments were performed, the results can be adopted as accurate and reliable. In numerous cases the writer has checked the values obtained with the calorimeter used by Mr. Hill with those obtained on the bomb calorimeter, commonly called Mahler's, and when the experiments have been carefully performed, no error exceeding 0.5% was found. This is about the limit of error in the operation of the instrument. As both of the instruments mentioned were standardized and operated by entirely different methods, the concurrence of results would indicate the substantial accuracy of both.

Table No. 30 gives the results of a determination of two samples of coal from Wills Creek, Ohio, made with the Carpenter calorimeter, together with a computation of the heating value from a complete chemical analysis. The results are interesting as showing the close agreement which may be obtained by entirely different methods.

Heat Value of Coal by Computation.—A complete analysis of this coal shows the following constituents of the volatile matter:

H 4.65%; O 9.40%; N 1.44%; S 1.43%.

Combining this analysis of the volatile matter with the approximate analysis we obtain:

Volatile matter, 20.31; moisture, 5.73; ash, 11.415; fixed carbon 46.625.

The heating value in B. T. U., corresponding to the analysis, may be calculated by the formula

$$h = 14\,650\ [C + 4.28\ (H - \frac{O}{8})] + 3\,740\ S$$

By substitution

$$h = 14\,650\ [66.935\% + 4.28\ (3.48\%)] + 53.6 = 12\,038.7$$

The calorimetric determination for this sample, which was used November 15th, 1898, gave 12 031 B. T. U. per pound of coal.

The close agreement of the computation with the calorimetric determination indicates the correctness of the determination as given.

Mr. Carpenter. The coals used on two different days from different carloads have heat values which differ from each other by less than 0.5%, and, hence, for practical purposes of comparison, they may be considered identical.

TABLE No. 30.—Approximate Analyses and Calorimetric Determinations, Mechanical Laboratory, Sibley College, Cornell University, November 15th, 1898.

	Coal used as received.	Dry coal. 100%	Combustible 100%
First sample.			
Moisture	5.68	0	0
Volatile matter	36.17	38.35	43.70
Fixed carbon	46.61	49.42	56.30
Ash	11.54	12.23	0
B. T. U., from calorimeter, per pound	12 016	12 739	14 515
Second sample.			
Moisture	5.78	0	0
Volatile matter	36.29	38.52	43.76
Fixed carbon	46.64	49.50	56.24
Ash	11.29	11.98	0
B. T. U., from calorimeter, per pound	12 047	12 807	14 526
Average.			
Moisture	5.73	0	0
Volatile matter	36.23	38.43	43.73
Fixed carbon	46.625	49.46	56.27
Ash	11.415	12.11	0
B. T. U. per pound	12 031	12 773	14 520

Note.—Coal from Wills Creek, Ohio, used in test of Wickes boiler at Wyandotte, November 15th, 1898. Sample of ash contained 32.9% of combustible, corresponding to 17.3% of ash in the test.

The writer has nothing to urge in relation to the advantages or disadvantages of any particular form of calorimeter, although doubtless much could be said in favor of or against any particular type, depending upon the standpoint from which the subject was viewed. He feels positive that the instrumental errors of the calorimeter used by Mr. Hill are as small in magnitude and as few in number as in those of any instrument made.

Mr. Hill. John W. Hill, M. Am. Soc. C. E. (by letter).—It should be understood that these tests, primarily, were to determine the relative physical value, under uniform conditions of service, of the steam coals available in the Cincinnati market, especially for use in the City Water Department, and the only way in which this could be accomplished satisfactorily was by firing the coals in the Department boilers by the Department firemen. To avoid interference with the regular daily operation of the Water-Works, the Board of Adminis-

tration required that the tests should be made at the Hunt Street Pumping Station, which has at all times a small surplus of power. It required that the coal should be fired in boilers which had no patented appliances especially adapted to one class or kind of coal, and that the steam should be used in performing the daily work of the pumping station. The author has no doubt that the Galloway boilers were not the best for all the coals tested, but believes that they were the best which could be had at the Hunt Street Pumping Station. With reference to the fitness of the boilers for this work, Mr. Kent evidently has overlooked the following paragraph on page 63: Mr. Hill.

"It is quite possible that the boilers used in these tests were not the best for all the coals tested, and that some kinds and grades of coal suffered accordingly; but that is a condition which perhaps will always be found true, for no form of boiler and furnace can be equally well adapted to every variety of bituminous (and semi-bituminous) coal, nor could changes be made in the boiler and furnace, and in the mode of firing, for the different kinds of coal, without raising a suspicion in the investigator's mind that by so doing he would be favoring one coal at the expense of another."

And on page 85:

"Reference to the physical efficiencies of these coals (Table No. 23), indicates that none of the coals was giving very high results, and some, undoubtedly, were worked at a disadvantage, although the evidences of this were not apparent at the time; but high efficiency of the low-grade coals cannot always be attained except in furnaces especially adapted for them. All the losses of heat by chimney draft, etc., should be limited to 30% of the ultimate heat value of the coal; and in designing boilers and furnaces, due regard should be had for the work to be performed and the coal to be used."

If the author understands Mr. Kent correctly he suggests that these different coals should have been tried under different boilers and by different firemen. Had this been done, it would have been impossible to have formed any correct estimate of the influence of boilers and firemen on the several coals, and would wholly have failed to answer the question submitted to the author by the Board of Administration. Undoubtedly, the proper method for the determination of the ultimate value of each coal would have been to test it with several kinds of boilers and furnaces, but this was wholly outside the scope of the author's authority or opportunities in these investigations.

Mr. Kent seems at first to commend the internally-fired boilers used in these trials, and then to object to them, because the furnace was not lined with fire-brick to secure better combustion of the volatile matter in the coals. This suggests the fire-box boilers for some kinds of coals, and boilers with brick furnaces for other kinds of coals; but since all these coals contained from 20 to 35% of volatile matter, any objection to the use of the fire-box boilers for the bituminous coals will apply, but possibly with less force, when used with semi-bituminous coals.

Mr. Hill. The author must take issue with Mr. Kent with reference to the rate of evaporation used in these tests. The Galloway boilers, and fire-box boilers generally, which have come under the author's observation, have been worked successfully at much higher rates than that adopted for these tests (5.33 lbs. of steam per square foot of heating surface per hour). Indeed, at the contract trial (from which some data are given on page 65), these boilers were worked at the rate of 7.57 lbs. of steam per square foot of water-heating surface per hour, from the temperature of feed water (193° Fahr.) to steam at 92.8 lbs. by gauge, or at a rate nearly 50% higher than that adopted by the author. Under this condition the boilers were said to have made, with Pittsburg Nut coal, 12.40 lbs. of steam from and at 212° Fahr. per pound of combustible. The trial mentioned was made by the late Mr. George H. Sellers, of Wilmington, Del.

It will be observed that, from Mr. Kent's point of view, the coal (Pittsburg) used by Mr. Sellers, by reason of its high percentage of volatile matter, was peculiarly unfitted for fire-box boilers.

With reference to the rate of work, the author does not believe that these coals should be burned at lower rates than shown in Column 6 of Tables Nos. 13, 14 and 15. In fact, since the trials were made, it has been thought that some of the coals were burned at too low a rate, per square foot of grate surface per hour, to obtain the best effects. It will be remembered that the boilers were worked at substantially the same rate of evaporation per square foot of heating surface per hour for all coals, and the extreme variation from the average rate was, with one exception, within 5 per cent.

The extreme ground, assumed by Mr. Kent, that the boilers were in an indifferent condition, driven too hard, and fired by inexperienced firemen, is wholly unwarranted. The boilers were in excellent condition, regularly cleaned at the end of each fourth test, and the firemen were the best employed by the City Water Department, and possibly were as expert in handling these coals as any firemen who were available for these tests.

The figures quoted by Mr. Kent on page 87 do not indicate what he terms "variable firing," but do indicate the variable quality of coal from the same dealers, and, presumably, from the same mine. Variations of 5 or 6% in the quality of coal from different sources and entries of the same mine are well known. In fact, in a given seam of coal a marked variation in heating power may be found at different heights in the seam.

Considering the experimental work of Professors Lord and Haas,* which shows, by the Mahler calorimeter, extreme variations (on com-

* *Loc. cit.*, page 73.

bustible) in different samples of coal from the same measures, the following percentages are found: Mr. Hill.

Pittsburg coal	5.3
Upper Freeport coal	5.5

Mr. Barrus, * from calorimetric tests of the same coals (on coal) obtained the following percentage variations:

George's Creek (Cumberland), coal	9.45
New River coal	4.20
Pocahontas coal	6.80
Clearfield Coal	8.20

The Acme coal and Coalburg coal are supposed to be from the same seam, and mined 12 miles apart. The difference between these coals by physical tests on combustible was 7.7 per cent. Again, the Cedar Grove and Belmont coals are mined by the same parties, on the same property, the seams being about 340 ft. (vertical) apart. The difference between these coals on combustible was 4.7 per cent. Part of these differences may be due to variable firing, but the larger portion is due to variability in the quality of the coal from different seams on the same property, or from the same seam on widely separated properties.

Mr. Kent's comparison of the coals from the Pittsburg mines, to demonstrate his belief that the firing in these tests was variable, is not altogether fair. The Brown mines and the Woods mine are separated about 20 miles, although supposed to be working the same seam of coal, and, in view of the reasonable uniformity of the several grades of coal from the respective mines, the comparison for variable firing (upon the assumption, which is not true, that the coal is of uniform quality throughout the seam) should be made upon the average performance of each coal, thus:

Source of Coal.	Pounds of steam per pound of Combustible from and at 212° Fahr.
Brown, four samples	9.42
Catsburg, four samples	9.46
Cincinnati, two samples	9.29
Woods, four samples	9.60
Monongah (West Virginia), one sample	9.58

All these coals are from the Pittsburg measures, and the combustible in the nut and slack is as good as in the higher grades of coal. Upon this showing, the extreme difference in the performance, and (by Mr. Kent's method of reasoning) variability of firing, is 3.23%; or, if the Cincinnati coal is omitted, the difference in these coals is less than 1.2 per cent. The Cincinnati coal, in both physical tests and by the calorimeter, falls distinctly under the other twelve samples from the Pittsburg seam.

* *Loc. cit.*, page 73.

Mr. Hill. The theory that all kinds of coal such as the author investigated will give a fairly uniform efficiency in any given type of boiler, is a novel proposition to the author, and is scarcely consistent with Mr. Kent's original suggestion, that these boilers were not well suited to all the coals tested. Experience has taught the author that the physical efficiency of a given coal will depend very much upon the adaptability of the boiler and furnace to that coal, and variations in efficiency must be anticipated when a single type of boiler and furnace is used with different kinds of coal.

Long before these investigations, the author had made many experiments on coals with the Thompson form of calorimeter, and had studied carefully all the available literature with reference to the use of the Berthelot bomb and other methods proposed for the determination of the heating power of fuels, and reached the conclusion that if the heat value of any substance had been accurately determined by any form of calorimeter, it was that of carbon; and if the heating value of pure carbon burning without ash is now known, that the Carpenter instrument would furnish as accurate and reliable results, when properly standardized, as the Berthelot or any other form of bomb, and, therefore, he adopted it for these investigations.

In order to check his work, as well as could be done from the available data, Table No. 20 was compiled. From this it appears that, with coal from a given locality, no startling differences occur between the work of the author and that of Messrs. Hale and Williams with a modified form of Berthelot bomb; of Professor Carpenter with his own instrument; and of Professors Lord and Haas with the Mahler bomb; and these differences are no greater than each experimenter probably has obtained in his own work.

The author sees no greater difficulty in standardizing the Carpenter calorimeter by pure carbon than in standardizing the Berthelot bomb by naphthalene, or any other substance of supposed known heat value. In one case, the heating power of pure carbon is assumed to have been accurately determined, and in the other case, the heating power of naphthalene is assumed to have been accurately determined. So far as combustion of the coal sample is concerned, there is no reason why it should not be as complete and perfect in the combustion chamber of the Carpenter instrument as in the combustion chamber of the bomb; nor does it appear to the author any more difficult to account for the total heat of combustion in the Carpenter instrument than in the Berthelot or Mahler bombs.

The use, in excess, of pure oxygen under pressure insures perfect and complete combustion, thus avoiding the usual loss due to imperfect combustion of the volatile matter in coal, when burned under steam boilers, and insuring by the presence of pure oxygen, the conversion of all combustible matter into carbon dioxid, sulphuric acid

and water. In using the Carpenter instrument, it is not convenient to collect the nitric acid obtained from the substance burned if nitrogen be present, nor is this necessary, because the heat liberated in converting the nitrogen into nitric acid seldom exceeds one-third of 1% of the total heat of the substance. Mr. Hill.

The advantage of the Carpenter over all other forms of coal calorimeter is in its simplicity of form and operation, and the readiness with which the notes may be reduced, thus avoiding the possible errors of complicated apparatus and tedious computation.

In reply to Mr. Kent's remark under Table No. 29, that in one instance (Cincinnati 3d Pool coal) the author's calorimetric work is not entitled to credit, Table No. 31, comparing the results obtained by the author, with the results obtained from the same kinds of coals by Professors Lord and Haas, and Professor Carpenter is offered.

TABLE No. 31.—B. T. U. Per Pound of Coal.

Kind of Coal.	Author.	Lord and Haas.	Carpenter.
New River	15 163	.....	.....
Pocahontas	15 133	14 906	15 094
Thacker	14 526	14 020	.....
Pittsburg (av.)	13 348	13 607	.....
Southern Ohio	12 733*	12 136†	.....
B. T. U. Per Pound of Combustible.			
New River	15 757	.....	.....
Pocahontas	15 456	15 750	15 870
Thacker	15 554	15 230	.....
Pittsburg (av.)	14 237	14 960	.....
Southern Ohio	14 218*	14 050†	.....

* Average Hocking Valley and Jackson coals.
† Hocking Valley coal.

The author does not know whether Mr. Kent considers the calorimetric work of the other investigators as entitled to credit, but if he does, it seems that the differences in heating value of the parallel coals tested are not so large as to suggest grave errors in the author's work. The extreme difference in the author's work on the Pittsburg (Pennsylvania) coals, by physical tests, is 3.23%, and by calorimeter (on combustible) 2.4%, which seems to indicate that Mr. Kent has made a mistake in rejecting the calorimetric value of Pittsburg-Cincinnati coal in his Table No. 29.

It may be observed in passing, that, in Table No. 29, Mr. Kent does not attempt to fit his curve to the Ohio coals having nearly the same percentage of fixed carbon in the net combustible as the Pittsburg and some of the Kanawha coals, doubtless for the reasons which he gives on page 90.

The heats given by Mr. Kent's curve for semi-bituminous coals containing about 80% of fixed carbon in the combustible, agree well with the results obtained by the calorimeter, but, for coals much lower

Mr. Hill. in fixed carbon, the differences are too great to admit of using the curve for either practical or scientific purposes.

For more than half a century it has been held that the heating value of bituminous and semi-bituminous coals was directly proportional to the percentage of fixed carbon in the combustible matter, although there is a limit to the application of this theory, because it is well known that the heat energy per unit of weight is less for the anthracite coals than for the semi-bituminous coals, like New River, Cumberland and Pocahontas, and a point is reached in the ratio of the fixed carbon to volatile matter which indicates the highest heat value theoretically attainable in coal. Above this point or ratio of fixed carbon to volatile matter, the thermal value of the coal declines until it reaches pure carbon, with a heating value of 14 650 B. T. U. per pound, and below this the heating value declines in an increasing ratio until the low value of 12 200 B. T. U. per pound is reached, with a combustible of which 50% is fixed carbon and 50% volatile matter.

The author does not share Mr. Bryan's fear that serious errors in the amount of coal charged to each trial, respectively, might have arisen from the method of starting and stopping the trials, considering that each of the four furnaces was fired from four to five times per hour. If all the furnaces had been charged with green coal just before a trial was started, and the grates left bare at the end of the trial, the maximum error in the coal charged would have been 1½%; but with the fires on each of the four grates burned down to the lowest point, while maintaining the steam pressure at the beginning and end of each trial, the probable error in the weight of coal charged must have been much less than the usual errors of observation. During the last 30 minutes, just before starting and stopping each trial, the firemen were under constant supervision, in order to avoid an excess or deficiency of coal on the grates, while maintaining the steam pressure and the speed of the pumping engine.*

Considering the temperature at which the feed-water was of necessity measured,† the meters seem to be the best device for the purpose. Under other conditions than those described in the paper, the author agrees with Mr. Bryan that it is preferable to weigh the water to the boilers.

With reference to cleaning the boilers before Trial No. 50‡ the paper neglects to state that the boilers were cleaned inside and out, and the grates and bridge-wall thoroughly overhauled and made equal to new.

In view of the relative value of New River and Pocahontas coals, it is singular that the Navy Department has not accorded the former coal the rating which it deserves. The author understands, unofficially,

* Refer to paragraph bottom of page 32 and top of page 33.

† Refer to paragraph bottom of page 33 and top of page 34.

‡ Refer to page 41. "Condition of Boilers."

that New River coal is rated at 90, Pocahontas coal being taken as the standard at 100; while, in fact, under the steam boilers by physical test, and upon the heating value by both coal and combustible, New River coal possesses a slight advantage over Pocahontas coal, and should be rated as high as the latter. Mr. Hill.

In burning all coals under steam boilers, only a fraction of the total heat goes into the water in the boilers. The remainder, which is lost for useful effect, is represented by the heat required to produce chimney draft, by evaporation of moisture in the air which passes through the furnaces and boilers, by evaporation of the moisture in the coal, by the latent heat in the gases of combustion, and by radiation and conduction of heat from the exposed surfaces of furnaces and boilers. Notes sufficient to show the losses of heat in detail were not taken during these tests, nor was it at the time thought necessary, because all the coals were burned in the same boilers, by the same firemen, and under uniform conditions of practice, and with "comparative" rather than "ultimate" effects in view.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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No. 855.

ON THE THEORY OF CONCRETE.

By GEORGE W. RAFTER, M. Am. Soc. C. E.

PRESENTED JUNE 7TH, 1899.

WITH DISCUSSION.

While employed on the original Genesee storage investigation in 1893, the author made a study of the available material for the construction of a high dam on the Genesee River, near Mount Morris, N. Y., the study including the fabrication of 173 blocks of concrete in cubes of 1 ft. each; the cement used including Cumming's Natural Buffalo cement, Newman's Akron, Bangs and Gaynor's Fayetteville, Norton's Rosendale, Millem's Wayland Portland and Saylor's Portland.

Tests were made, at the same time, of the voids in the local sand and in the different stones used as aggregates, and also of the tensile strength of the several cements. The results of these tests may be found in Appendix E of the Annual Report of the State Engineer and Surveyor of New York, for the fiscal year ending September 30th, 1894.

In 1896, further tests were made, which included the fabrication of 544 additional concrete foot-cubes, the cements tested including Whitmore, Rauber and Vicinus' Genesee brand, Millem's Wayland, the Glens Falls Portland Cement Company's Iron-Clad brand, the Empire Portland Cement Company's Empire brand, the Silica Portland

Cement Company's Champion brand, Cumming's Buffalo Portland and Cumming's Buffalo Natural. The detail of this second set of tests may be found in the Appendix to the Annual Report of the State Engineer and Surveyor of New York, for the fiscal year ending September 30th, 1897, to which reference may be made.

Tables Nos. 1 and 2 embody certain results of the second series of tests made in 1896, which, to a large extent, are not included in the report to the State Engineer, and which have seemed to the author to be of possible interest to the members of the Society. They are, therefore, submitted for comment and criticism.

The following explanations are condensed from the author's report to the State Engineer on these tests.

The leading ideas of the tests made in 1896 were: (1) To determine the strength of concretes fabricated with either dry mortars, plastic mortars or mortars with an excess of water; (2) to determine the effect on concrete of exposure to the weather; and (3) to determine the relative value for use in concrete of the Portland cements manufactured in the State of New York. The various qualities of mortar used are designated by the words "dry," "plastic" and "excess." In the dry blocks the mortar was only a little more moist than damp earth. In the plastic blocks the mortar was of the ordinary consistency used by masons. In the excess blocks the water was so far in excess that the concrete quaked like liver, under moderate ramming.

The serial number of Table No. 1 represents a complete series as designated by the Arabic numeral and the letters "a," "b" and "c" of Table No. 1 of the concrete report.* Thus, by way of illustration, Series 1 of Table No. 1 is a combination of blocks 1, 1a, 1b and 1c of Table No. 1 of the report; Series 2 of Table No. 1 represents blocks 2, 2a, 2b and 2c of Table No. 1 of the report, and so on. The headings of the several columns of Table No. 1 are self-explanatory. The blocks were broken on the large Government testing machine at the Watertown Arsenal.

* On referring to Table No. 1 of the report on concrete in the Annual Report of the State Engineer and Surveyor of New York, for 1897, it will be observed that the blocks were made in series of four. Those designated by the Arabic numeral were placed in water when taken from the molds, and allowed to remain until December 1st, 1896, when they were buried in sand, where they remained until taken out for shipment January 10th and 11th, 1898. Blocks marked with the Arabic numeral and the letter "a" stood in a cool cellar from the time of taking from the molds until shipped. Blocks marked with the Arabic numeral and the letter "b" were placed in open air when taken from the molds, and exposed to the weather until shipped. Blocks marked with the Arabic numeral and the letter "c" were placed in open weather when taken from the molds, covered with burlap and wet with water several times a day until November 1st, 1896, after which time they took the weather as it came, until shipped.

The formulas by which the quantities in columns (14), (17), (18) and (19) have been calculated, are as follows:

FORMULA FOR COLUMN (14).

Dry cement used in batch of concrete, : Mortar used in batch of concrete, :: Dry cement used in mortar, : Cubic feet of mortar made;

or, Column (14), : Column (11), : : Column (7), : Column (9).

FORMULA FOR COLUMN (17).

Number volumes mortar in concrete for one of cement, : One volume of cement, :: Mortar originally made, : Cement used in batch of mortar;

or, Column (17), : 1, : : Column (9), : Column (7).

FORMULA FOR COLUMN (18).

Number volumes stone used in concrete for one of cement, : One volume of cement, :: Stone used in batch of concrete, : Dry cement used in batch of concrete;

or, Column (18), : 1, : : Column (12), : Column (14).

FORMULA FOR COLUMN (19).

Number volumes concrete made for one volume of cement used, : One volume of cement, :: Concrete actually made, : Cement used in batch of concrete;

or, Column (19), : 1, : : Column (13), : Column (14).

In reference to column (10) of Table No. 1 the author holds that an erroneous nomenclature has considerably obscured the theory and practice of concrete work, so much so that very few contractors and engineers have other than exceedingly general ideas as to the real theory of what is in effect a very simple matter, there being, indeed, very little necessary except to insure the filling of the voids in the aggregate material with mortar. In the author's opinion, therefore, the proper form of expressing concrete relations is not by parts 1: 2: 5; 1: 3: 7, etc.; but rather to express the volume of mortar as a percentage of the volume of stone, the percentage figure used being large enough to represent a little more than the voids in the stone after thorough ramming. The chief object of this paper, therefore, is to point out: (1) The relation existing between the old nomenclature and the new, as proposed by the author, and, (2) to present a brief *résumé* of the results given in detail in the report to the State Engineer and Surveyor.

In Table No. 2 the series have been arranged for the different proportions of cement experimented upon with reference to the consistency

of the mortar, the proportion of sand and cement and the percentage of mortar actually used in each series. Thus, in the first part of Table No. 2 we find compared the Genesee, Wayland and Champion cements for dry mortars with the proportion of sand to cement 1 to 1 and the mortar 33% of the aggregate. In the second division of Table No. 2 the same cements are compared with reference to plastic mortars, other conditions remaining the same; and in the third division they are compared with reference to excess mortars, other conditions remaining the same, and so on, through this table to the summary.

Column 1 of Table No. 2 gives the serial number, the figures there appearing corresponding not only to Table No. 1 of this paper, but also to Table No. 1 of the report to the State Engineer and Surveyor; columns (3), (17), (18), (19) and (20) of this table are the same as the correspondingly numbered columns of Table No. 1 of this paper. Column A of Table No. 2 gives the percentage of shrinkage due to ramming. Thus, column (18) represents the actual quantity of stone used, and column (19) the actual quantity of concrete obtained from mortar and stone as per columns (17) and (18). In the case of the Genesee, Wayland and Champion brands, 1 to 1 mortar, 33% in the aggregate was tried; this shrinkage is found to be, as a mean, 9.3%; for the same cements in plastic mortar, other conditions the same, the shrinkage is 9.1%, and for excess mortars, 9.2 per cent. Broadly, it may be said, therefore, that for 1 to 1 mortars, using 33% of the aggregate, with thorough ramming, about 9% shrinkage may be expected. In the same way, with 1 to 1 mortars, 40% of the aggregate, about 7% shrinkage may be expected. Similar relations, which may be traced through the entire series of tests, are conveniently summarized in Table No. 3.

It will be noticed in studying Table No. 2, that a few of the results are somewhat irregular. On this point the author can only say that very thorough precautions were taken to have the results reasonably accurate. Probably the irregular results are largely due to occasional variations in the quality of the broken stone used for these tests. There were also some variations in the sand, the weight per cubic foot varying from about 86.5 to 93.5 pounds. So far as the author can see, there is no way of avoiding such variations except by determining the voids in the aggregate material for every series of blocks and by actually drying the sand to a uniform consistency. It may be pointed out, however, that such procedure would defeat the

chief object for which these tests were made, namely, to work out methods of procedure which could be conveniently applied in practice, without unduly adding to the cost of concrete. From this point of view the author considers the tests here discussed as exceedingly satisfactory. They are in the fullest sense practical results which may be carried out on work by anybody who will give the theory of concrete moderate study.

It may be further pointed out that columns (15) to (19) of Table No. 1 furnish ready means of computing the proportionate parts of the ingredients for any given volume of concrete.

Many other details could be given, but inasmuch as the object of this paper is to merely call attention to the real points at issue in concrete fabrication and not to repeat statements already in print in the place cited, all details other than those actually necessary for an understanding of the tables of this paper have been omitted. The results are, therefore, submitted in the hope that they may finally lead to more rational concrete practice.

TABLE No. 1.—DATA OF CONCRETE TESTS.

Serial number.	Consistency of mortar.	Mortar.							Concrete.					Proportional Parts by Volume (as per preceding data).					Mean ultimate strength of each series of blocks in pounds per square inch.
		Brand of cement.	Parts of cement by volume.	Parts of sand by volume.	Water used, in pounds per cubic foot of mortar.	Cement actually used, in cubic feet.	Sand actually used, in cubic feet.	Mortar actually made from water, cement and sand, as per columns (6), (7) and (8).	Stone: Ratio to mortar. Percentage.	Mortar actually used in making the quantities of concrete shown in column (13). Cubic feet.	Stone actually used. Cubic feet.	Concrete actually made from mortar and stone as per columns (11) and (12). Cubic feet.	Dry cement actually used in batch of concrete as per column (13). Cubic feet.	Cement.	Sand.	Mortar.	Stone.	Concrete.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
1	Dry.	Genesee.	1	2	16.6	1	2	2.40	33	1.67	5	4.83	0.70	1	2	2.40	7.14	6.90	2 852
2	Plastic.	"	1	3	.....	1	3	3.25	33	1.67	5	4.50	0.51	1	3	3.25	9.80	8.82	2 616
3	Excess.	"	1	4	20.7	1	4	3.83	33	1.67	5	4.10	0.43	1	4	3.83	11.62	9.54	2 416
4	Dry	"	1	2	17.0	1	2	2.45	40	2.00	5	5.20	0.81	1	2	2.45	6.17	6.42	2 963
5	"	"	1	3	18.0	$\frac{2}{3}$	2	2.17	40	2.00	5	4.82	0.71	1	3	3.25	8.12	6.78	2 423
6	Excess.	"	1	4	22.9	$\frac{2}{3}$	$2\frac{2}{3}$	2.83	40	2.00	5	4.82	0.58	1	4	4.25	10.63	8.31	2 132
7	Dry.	"	1	1	18.4	1	1	1.58	33	1.67	5	.....	1.06	1	1	1.58	4.71	.....	4 130
8	"	"	1	3	17.6	$\frac{2}{3}$	2	1.98	33	1.67	5	4.83	0.56	1	2	2.97	8.93	8.63	3 065
9	"	"	1	4	17.0	$\frac{1}{2}$	2	2.00	33	1.67	5	4.83	0.41	1	4	4.00	12.19	11.78	2 059
10	"	"	1	5	19.1	$\frac{1}{2}$	$2\frac{1}{2}$	2.46	33	1.67	5	4.83	0.34	1	5	4.92	14.70	14.20	1 664
11	"	"	1	6	17.5	$\frac{1}{2}$	3	2.79	33	1.67	5	4.92	0.30	1	6	5.58	16.66	16.40	1 427
12	Plastic.	"	1	1	.....	1	1	1.75	33	1.67	5	4.50	0.90	1	1	1.75	5.25	4.77	3 505
13	"	"	1	3	23.1	$\frac{2}{3}$	2	2.25	33	1.5	4.5	4.17	0.44	1	3	3.38	10.14	8.70	2 311
14	"	"	1	4	22.0	$\frac{1}{2}$	2	2.14	33	1.5	4.5	4.08	0.35	1	4	4.28	12.86	11.70	2 036
15	"	"	1	5	22.4	$\frac{1}{2}$	$2\frac{1}{2}$	2.50	33	1.5	4.5	4.12	0.30	1	5	5.00	15.00	13.73	1 801
16	Excess	"	1	1	24.5	1	1	1.66	33	1.5	4.5	4.08	0.90	1	1	1.66	4.98	4.54	3 622
17	"	"	1	2	24.1	$\frac{3}{4}$	$1\frac{1}{2}$	1.66	33	1.5	4.5	4.12	0.68	1	2	2.21	6.62	6.06	2 524
18	"	"	1	3	20.0	$\frac{2}{3}$	2	2.16	33	1.5	4.5	4.12	0.46	1	3	3.24	9.78	8.96	2 255
19	Dry.	"	1	2	19.0	1	2	2.58	40	1.8	4.5	4.58	0.70	1	2	2.58	6.43	6.54	2 988
20	"	"	1	3	19.7	$\frac{2}{3}$	2	2.00	40	1.8	4.5	4.33	0.60	1	3	3.00	7.50	7.22	2 213
21	"	"	1	4	18.0	$\frac{1}{2}$	2	2.17	40	1.8	4.5	.....	0.41	1	4	4.34	10.97	.....	1 972
22	Plastic.	"	1	2	20.4	1	2	2.25	40	1.8	4.5	4.21	0.80	1	2	2.25	5.62	5.26	2 962
23	"	"	1	3	21.3	$\frac{2}{3}$	2	2.17	40	1.8	4.5	4.17	0.56	1	3	3.26	8.03	7.44	2 054
24	"	"	1	4	21.3	$\frac{1}{2}$	2	2.17	40	1.8	4.5	4.17	0.41	1	4	4.34	10.90	10.16	1 828

TABLE No. 1—(*Concluded*).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
70	Dry.	Ironclad.	1	4	21.3	$\frac{1}{2}$	2	2.06	40	1.8	4.5	4.17	0.43	1	4	4.12	10.46	9.70	1 652
71	Plastic.	"	1	2	.....	1	2	2.69	40	1.8	4.5	4.17	0.67	1	2	2.69	6.71	6.22	2 994
72	"	"	1	3	.....	$\frac{3}{4}$	2	2.25	40	1.8	4.5	4.17	0.54	1	3	3.38	8.38	7.70	1 889
73	"	"	1	4	.....	$\frac{1}{2}$	2	2.23	40	1.8	4.5	.....	0.40	1	4	4.46	11.25	.....	1 544
74	Excess.	"	1	2	.....	1	2	2.62	40	1.8	4.5	4.17	0.69	1	2	2.62	6.51	6.02	2 866
75	"	"	1	3	.....	$\frac{3}{4}$	2	2.27	40	1.8	4.5	4.17	0.53	1	3	3.40	8.50	7.86	2 083
76	"	"	1	4	.....	$\frac{1}{2}$	2	2.27	40	1.8	4.5	4.17	0.40	1	4	4.54	11.25	10.43	1 508
77	Dry.	Empire.	1	2	20.6	1	$1\frac{1}{2}$	1.87	33	1.5	4.5	4.08	0.60	1	2	2.49	7.50	6.80	3 265
78	"	"	1	3	20.0	$\frac{3}{4}$	2	2.17	33	1.5	4.5	4.08	0.46	1	3	3.25	9.78	8.62	1 992
79	"	"	1	4	18.4	$\frac{1}{2}$	2	2.13	33	1.5	4.5	4.08	0.35	1	4	4.26	12.85	11.05	1 666
80	Plastic.	"	1	2	.....	1	$1\frac{1}{2}$	1.79	33	1.5	4.5	4.08	0.62	1	2	2.39	6.71	6.05	2 529
81	"	"	1	3	.....	$\frac{3}{4}$	2	2.29	33	1.5	4.5	4.08	0.44	1	3	3.44	10.23	9.26	2 014
82	"	"	1	4	.....	$\frac{1}{2}$	2	2.19	33	1.5	4.5	4.08	0.34	1	4	4.38	13.24	12.00	1 523
83	Excess.	"	1	2	21.7	1	$1\frac{1}{2}$	1.97	33	1.5	4.5	4.08	0.60	1	2	2.63	7.50	6.80	2 899
84	"	"	1	3	21.7	$\frac{3}{4}$	2	2.29	33	1.5	4.5	4.08	0.44	1	3	3.44	10.23	9.26	1 898
85	"	"	1	4	22.3	$\frac{1}{2}$	2	2.21	33	1.5	4.5	4.08	0.34	1	4	4.42	13.24	12.00	1 488
86	Dry.	"	1	2	20.0	1	2	2.52	40	1.8	4.5	4.17	0.71	1	2	2.52	6.33	5.87	3 413
87	"	"	1	3	18.2	$\frac{3}{4}$	2	2.15	40	1.8	4.5	4.17	0.56	1	3	3.23	8.03	7.43	2 376
88	"	"	1	4	18.1	$\frac{1}{2}$	2	2.12	40	1.8	4.5	4.17	0.42	1	4	4.24	10.71	9.91	1 806
89	Plastic.	"	1	2	20.7	1	2	2.56	40	1.8	4.5	4.17	0.70	1	2	2.56	6.43	5.96	3 101
90	"	"	1	3	20.9	$\frac{3}{4}$	2	2.21	40	1.8	4.5	4.20	0.54	1	3	3.32	8.33	7.75	2 123
91	"	"	1	4	20.5	$\frac{1}{2}$	2	2.13	40	1.8	4.5	4.17	0.42	1	4	4.26	10.71	9.93	1 726
92	Excess.	"	1	2	23.4	1	2	2.56	40	1.8	4.5	4.17	0.70	1	2	2.56	6.43	5.96	2 729
93	"	"	1	3	23.7	$\frac{3}{4}$	2	2.29	40	1.8	4.5	4.17	0.52	1	3	3.41	8.58	8.00	1 913
94	"	"	1	4	22.9	$\frac{1}{2}$	2	2.17	40	1.8	4.5	.....	0.41	1	4	4.34	10.97	.....	1 472
95	Dry.	Champion.	1	1	19.1	$1\frac{1}{2}$	$1\frac{1}{2}$	2.46	33	1.5	4.5	4.08	0.90	1	1	1.64	5.00	4.53	2 659
96	"	"	1	2	18.8	1	2	2.37	33	1.5	4.5	4.17	0.63	1	2	2.37	7.14	6.61	2 289
97	"	"	1	3	17.3	$\frac{3}{4}$	2	2.13	33	1.5	4.5	4.17	0.47	1	3	3.17	9.59	8.90	1 401
98	Plastic.	"	1	1	21.3	$1\frac{1}{2}$	$1\frac{1}{2}$	2.37	33	1.5	4.5	4.08	0.94	1	1	1.58	4.79	4.35	2 503
99	"	"	1	2	20.0	1	2	2.42	33	1.5	4.5	4.08	0.62	1	2	2.42	7.25	6.58	2 146
100	"	"	1	3	20.0	$\frac{3}{4}$	2	2.17	33	1.5	4.5	4.08	0.46	1	3	3.26	9.78	8.87	1 337
101	Excess.	"	1	1	22.2	$1\frac{1}{4}$	$1\frac{1}{4}$	2.13	33	1.5	4.5	4.08	0.88	1	1	1.70	5.11	4.64	2 279
102	"	"	1	2	21.4	1	2	2.48	33	1.5	4.5	4.08	0.60	1	2	2.48	7.50	6.80	1 862
103	"	"	1	3	24.2	$\frac{3}{4}$	2	2.00	33	1.5	4.5	4.08	0.50	1	3	3.00	9.00	8.16	1 168
104	Dry.	"	1	1	18.5	$1\frac{1}{4}$	$1\frac{1}{4}$	1.96	40	1.8	4.5	4.25	1.15	1	1	1.57	3.90	3.70	3 125
105	"	"	1	2	19.4	1	2	2.06	40	1.8	4.5	4.25	0.87	1	2	2.06	5.18	4.90	2 204
106	"	"	1	3	19.0	$\frac{3}{4}$	2	2.13	40	1.8	4.5	4.17	0.56	1	3	3.20	8.03	7.44	1 400
107	Plastic.	"	1	1	22.1	$1\frac{1}{4}$	$1\frac{1}{4}$	2.08	40	1.8	4.5	4.08	1.08	1	1	1.66	4.16	3.77	2 603
108	"	"	1	2	.....	1	2	2.44	40	1.8	4.5	4.17	0.74	1	2	2.44	6.08	5.62	2 037
109	"	"	1	3	20.0	$\frac{3}{4}$	2	2.19	40	1.8	4.5	4.17	0.55	1	3	3.29	8.18	7.58	1 193
110	Excess.	"	1	1	22.3	$1\frac{1}{4}$	$1\frac{1}{4}$	2.10	40	1.8	4.5	4.17	1.07	1	1	1.68	4.20	4.00	2 196
111	"	"	1	2	21.0	1	2	2.48	40	1.8	4.5	4.17	0.73	1	2	2.48	6.18	5.71	1 711
112	"	"	1	3	22.2	$\frac{3}{4}$	2	2.29	40	1.8	4.5	.....	0.52	1	3	3.44	8.65	.....	1 127
113	Plastic.	Buffalo Por'd	1	2	19.6	1	2	2.54	40	1.8	4.5	4.17	0.70	1	2	2.54	6.43	5.96	1 717
114	"	"	1	3	20.0	$\frac{3}{4}$	2	2.19	40	1.8	4.5	4.17	0.55	1	3	3.29	8.18	7.58	958

TABLE No. 1—(*Continued*).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
25	Excess.	Genesee.	1	1	24.0	1¼	2	2.50	40	1.8	4.5	4.17	1.08	1	1	1.67	4.16	3.85	3 191
26	"	"	1	2	22.9	1	2	2.75	40	1.8	4.5	4.08	0.66	1	2	2.75	6.82	6.19	2 544
27	"	"	1	3	20.8	2	2	2.33	40	1.8	4.5	4.08	0.51	1	3	3.50	8.80	8.00	2 107
28	"	"	1	4	21.0	2	2	2.08	40	1.8	4.5	.....	0.43	1	4	4.16	10.40	.....	1 846
29	Dry.	Wayland.	1	1	.....	1¼	1½	1.87	33	1.5	4.5	4.08	1.00	1	1	1.50	4.50	4.08	4 267
30	"	"	1	2	.....	1½	1½	1.64	33	1.5	4.5	4.17	0.68	1	2	2.19	6.62	6.12	2 888
31	"	"	1	3	18.8	2	2	2.08	33	1.5	4.5	4.17	0.48	1	3	3.12	9.38	8.69	2 056
32	"	"	1	4	17.7	2	2	2.12	33	1.5	4.5	4.08	0.35	1	4	4.24	12.86	11.66	1 810
33	"	"	1	5	17.1	2½	2	2.58	33	1.5	4.5	4.17	0.29	1	5	5.16	15.41	14.39	1 537
34	Plastic.	"	1	1	19.0	1	1	2.17	33	1.5	4.5	4.08	0.69	1	1	2.17	6.50	5.90	4 072
35	"	"	1	2	21.3	1½	2	1.83	33	1.5	4.5	4.12	0.61	1	2	2.44	7.39	6.75	2 777
36	"	"	1	3	20.0	2	2	2.12	33	1.5	4.5	4.08	0.47	1	3	3.18	9.57	8.68	2 207
37	"	"	1	4	21.2	2	2	2.12	33	1.5	4.5	4.08	0.35	1	4	4.24	12.80	11.34	1 600
38	"	"	1	5	22.0	2½	2	2.50	33	1.5	4.5	4.08	0.30	1	5	5.00	15.00	13.60	1 586
39	Excess.	"	1	1	24.3	1	1	1.75	33	1.5	4.5	4.08	0.86	1	1	1.75	5.28	4.75	3 764
40	"	"	1	2	22.1	1	2	2.04	33	1.5	4.5	4.08	0.73	1	2	2.04	6.23	5.55	2 847
41	"	"	1	3	22.4	2	2	2.08	33	1.5	4.5	4.08	0.48	1	3	3.12	9.37	8.50	1 723
42	"	"	1	4	22.5	2	2	2.12	33	1.5	4.5	4.08	0.35	1	4	4.24	12.71	11.70	1 767
43	"	"	1	5	23.0	2½	2	2.54	33	1.5	4.5	4.08	0.30	1	5	5.08	15.20	13.60	1 441
44	Dry.	"	1	1	21.0	1½	2	2.58	40	1.8	4.5	4.17	1.06	1	1	1.72	4.30	3.93	3 966
45	"	"	1	2	21.6	1	2	2.54	40	1.8	4.5	4.17	0.71	1	2	2.54	6.35	5.87	3 404
46	"	"	1	3	20.8	2	2	2.17	40	1.8	4.5	4.17	0.55	1	3	3.26	8.18	7.59	2 179
47	"	"	1	4	20.8	2	2	2.12	40	1.8	4.5	4.17	0.42	1	4	4.24	10.71	9.92	1 671
48	"	"	1	5	20.7	2½	2	2.21	40	1.8	4.5	.....	0.40	1	5	4.42	11.25	.....	1 559
49	Plastic.	"	1	1	22.6	1½	2	2.50	40	1.8	4.5	4.17	1 08	1	1	1.67	4.12	3.86	4 123
50	"	"	1	2	21.8	1	2	2.54	40	1.8	4.5	4.17	0.70	1	2	2.54	6.43	5.96	2 960
51	"	"	1	3	21.6	2	2	2.21	40	1.8	4.5	4.17	0.54	1	3	3.32	8.29	7.68	2 027
52	"	"	1	4	21.4	2	2	2.17	40	1.8	4.5	4.17	0.41	1	4	4.34	10.97	10.17	1 750
53	"	"	1	5	.....	2½	2	2.50	40	1.8	4.5	4.17	0.36	1	5	5.00	12.50	11.56	1 465
54	Excess.	"	1	1	22.9	1½	1½	2.62	40	1.8	4.5	4.17	1.03	1	1	1.75	4.37	4.05	3 256
55	"	"	1	2	22.9	1	2	2.58	40	1.8	4.5	4.17	0.70	1	2	2.58	6.43	5.96	3 168
56	"	"	1	3	22.2	2	2	2.25	40	1.8	4.5	4.17	0.54	1	3	3.38	8.34	7.75	2 016
57	"	"	1	4	22.0	2	2	2.17	40	1.8	4.5	4.17	0.41	1	4	4.34	10.97	10.17	1 670
58	"	"	1	5	.....	2½	2	2.62	40	1.8	4.5	.....	0.35	1	5	5.24	12.99	.....	1 400
59	Dry.	Ironclad.	1	2	20.0	1½	2	1.98	33	1.5	4.5	4.08	0.56	1	2	2.64	8.04	7.28	2 777
60	"	"	1	3	20.8	2	2	2.17	33	1.5	4.5	4.17	0.46	1	3	3.26	9.78	9.06	1 501
61	"	"	1	4	21.2	2	2	2.12	33	1.5	4.5	4.17	0.35	1	4	4.24	12.85	11.90	1 412
62	Plastic.	"	1	2	22.0	1½	2	1.92	33	1.5	4.5	4.08	0.58	1	2	2.56	7.76	7.04	2 552
63	"	"	1	3	22.7	2	2	2.21	33	1.5	4.5	4.08	0.45	1	3	3.31	10.00	9.06	1 750
64	"	"	1	4	22.4	2	2	2.12	33	1.5	4.5	4.08	0.35	1	4	4.24	12.85	11.62	1 434
65	Excess.	"	1	2	24.0	1¼	2	1.80	33	1.5	4.5	4.08	0.62	1	2	2.40	7.25	6.58	2 436
66	"	"	1	3	.....	2	2	2.29	33	1.5	4.5	4.08	0.44	1	3	3.44	10.23	9.26	1 718
67	"	"	1	4	23.7	2	2	2.50	33	1.5	4.5	4.17	0.30	1	4	5.00	15.00	13.90	1 594
68	Dry.	Ironclad.	1	2	20.4	1	2	2.50	40	1.8	4.5	4.17	0.72	1	2	2.50	6.25	5.78	3 217
69	"	"	1	3	21.5	2	2	2.21	40	1.8	4.5	4.17	0.54	1	3	3.32	8.32	7.73	1 929

TABLE No. 2.—Concrete Blocks with Mortar 33% of the Aggregate, in Comparison with Blocks in which the Mortar was 40% of the Aggregate.

Serial number.	Brand of cement.	Mortar.	Stone.	Concrete.	Percentage of shrinkage.	Mean ultimate strength of each series, in pounds per square inch.
(1)	(3)	(17)	(18)	(19)	(A)	(20)
		Dry 1 : 1. 33% Mortar.				
7	Genesee	1.58	4.71	.....	.....	4 130
29	Wayland	1.50	4.50	4.08	.....	4 267
95	Champion	1.64	5.00	4.53	.....	2 659
.....	Means	1.57	4.74	4.30	9.3	.....
		Plastic 1 : 1. 33% Mortar.				
12	Genesee	1.75	5.25	4.77	.....	3 505
34	Wayland	2.17	6.50	5.90	.....	4 072
98	Champion	1.58	4.79	4.35	.....	2 593
.....	Means	1.83	5.51	5.01	9.1	.....
		Excess 1 : 1. 33% Mortar.				
16	Genesee	1.66	4.98	4.54	.....	3 622
39	Wayland	1.73	5.23	4.75	.....	3 764
101	Champion	1.70	5.11	4.64	.....	2 279
.....	Means	1.70	5.11	4.64	9.2	.....
		Dry 1 : 1. 40% Mortar.				
44	Wayland	1.72	4.30	3.93	.....	3 966
104	Champion	1.57	3.90	3.70	.....	3 125
.....	Means	.....	4.10	3.82	6.8	.....
		Plastic 1 : 1. 40% Mortar.				
49	Wayland	1.67	4.12	3.86	.....	4 123
107	Champion	1.66	4.16	3.77	.....	2 603
.....	Means	1.66	4.14	3.82	7.7	.....
		Excess 1 : 1. 40% Mortar				
25	Genesee	1.67	4.16	3.85	.....	3 191
54	Wayland	1.75	4.37	4.05	.....	3 256
110	Champion	1.68	4.20	4.00	.....	2 196
.....	Means	1.70	4.24	3.97	6.4	.....
		Dry 1 : 2. 33% Mortar.				
1	Genesee	2.40	7.14	6.90	.....	2 852
30	Wayland	2.19	6.62	6.12	.....	2 888
59	Ironclad	2.64	8.04	7.28	.....	2 777
77	Empire	2.49	7.50	6.80	.....	3 265
96	Champion	2.37	7.14	6.61	.....	2 289
.....	Means	2.42	7.29	6.74	7.5	.....

TABLE No. 2—(*Continued*).

(1)	(3)	(17)	(18)	(19)	(A)	(20)
		PLASTIC 1 : 2. 33% MORTAR.				
35.....	Wayland	2.44	7.39	6.75	.....	2 777
62.....	Ironclad.....	2.56	7.76	7.04	.....	2 552
80.....	Empire.....	2.39	6.71	6.09	.....	2 529
99.....	Champion.....	2.42	7.25	6.58	.....	2 146
.....	Means.....	2.45	7.28	6.62	9.1	.....
		EXCESS 1 : 2. 33% MORTAR.				
17.....	Genesee	2.21	6.62	6.06	.....	2 524
40.....	Wayland	2.04	6.23	5.55	.....	2 847
65.....	Ironclad.....	2.40	7.25	6.58	.....	2 436
83.....	Empire.....	2.63	7.50	6.80	.....	2 899
102.....	Champion.....	2.48	7.50	6.80	.....	1 862
.....	Means.....	2.35	7.02	6.36	9.4	.....
		DRY 1 : 2. 40% MORTAR.				
19.....	Genesee.....	2.58	6.43	6.54	.....	2 988
45.....	Wayland	2.54	6.35	5.87	.....	3 404
68.....	Ironclad.....	2.50	6.25	5.78	.....	3 217
86.....	Empire.....	2.52	6.33	5.87	.....	3 413
105.....	Champion.....	2.06	5.18	4.90	.....	2 204
.....	Means.....	2.44	6.11	5.79	5.3	.....
		PLASTIC 1 : 2. 40% MORTAR.				
22.....	Genesee.....	2.25	5.62	5.26	.....	2 962
50.....	Wayland	2.54	6.43	5.96	.....	2 960
71.....	Ironclad.....	2.69	6.71	6.22	.....	2 994
89.....	Empire.....	2.56	6.43	5.96	.....	3 101
108.....	Champion.....	2.44	6.08	5.62	.....	2 037
113.....	Buffalo Portland..	2.54	6.43	5.96	.....	1 717
.....	Means.....	2.50	6.28	5.83	7.1	.....
		EXCESS 1 : 2. 40% MORTAR.				
26.....	Genesee.....	2.75	6.82	6.19	.....	2 544
55.....	Wayland	2.58	6.43	5.96	.....	3 168
74.....	Ironclad.....	2.62	6.51	6.03	.....	2 866
92.....	Empire.	2.56	6.43	5.96	.....	2 729
111.....	Champion.....	2.48	6.18	5.71	.....	1 711
.....	Means.....	2.60	6.47	5.97	7.1	.....
		DRY 1 : 3. 33% MORTAR.				
8.....	Genesee.....	2.97	8.93	8.63	.....	3 065
31.....	Wayland	3.12	9.38	8.69	.....	2 056
60.....	Ironclad.....	3.26	9.78	9.06	.....	1 501
78.....	Empire.....	3.25	9.78	8.62	.....	1 992
97.....	Champion.....	3.17	9.59	8.90	.....	1 401
.....	Means.....	3.15	9.49	8.78	7.5	.....
		PLASTIC 1 : 3. 33% MORTAR.				
13.....	Genesee.....	3.38	10.14	8.70	.....	2 311
36.....	Wayland	3.18	9.57	8.62	.....	2 207
63.....	Ironclad.....	3.31	10.00	9.06	.....	1 750
81.....	Empire.....	3.44	10.23	9.27	.....	2 014
100.....	Champion.....	3.26	9.78	8.87	.....	1 337
2.....	Genesee	3.25	9.80	8.82	.....	2 616
.....	Means	3.30	9.92	8.89	10.4	.....

TABLE No. 2—(*Continued*).

(1)	(3)	(17)	(18)	(19)	(A)	(20)
		EXCESS 1 : 3. 33% MORTAR.				
18.....	Genesee	3.24	9.78	8.96	.....	2 255
41.....	Wayland.....	3.12	9.37	8.50	.....	1 723
66.....	Ironclad.....	3.44	10.23	9.26	.....	1 718
84.....	Empire	3.44	10.23	9.26	.....	1 898
103.....	Champion.....	3.00	9.00	8.16	.....	1 168
.....	Means.....	3.25	9.72	8.83	9.2	.....
		DRY 1 : 3. 40% MORTAR.				
20.....	Genesee	3.00	7.50	7.22	.....	2 213
46.....	Wayland	3.26	8.18	7.59	.....	2 179
69.....	Ironclad.....	3.32	8.32	7.73	.....	1 929
87.....	Empire	3.23	8.03	7.43	.....	2 376
106.....	Champion	3.20	8.03	7.44	.....	1 400
5.....	Genesee.....	3.25	8.12	6.78	.....	2 423
.....	Means.....	3.21	8.03	7.36	8.4	.....
		PLASTIC 1 : 3. 40% MORTAR				
23.....	Genesee.....	3.26	8.03	7.44	.....	2 054
51.....	Wayland.....	3.32	8.29	7.68	.....	2 027
72.....	Ironclad.....	3.38	8.38	7.70	.....	1 839
90.....	Empire.....	3.32	8.33	7.75	.....	2 023
109.....	Champion.....	3.29	8.18	7.58	.....	1 193
114.....	Buffalo Portland..	3.29	8.18	7.58	.....	958
.....	Means	3.31	8.23	7.62	7.3	.....
		EXCESS 1 : 3. 40% MORTAR.				
27.....	Genesee.....	3.50	8.80	8.00	.....	2 107
56.....	Wayland.....	3.38	8.34	7.75	.....	2 016
75.....	Ironclad.....	3.40	8.50	7.86	.....	2 083
93.....	Empire.....	3.44	8.58	8.00	.....	1 913
112.....	Champion.....	3.44	8.65	.....	.....	1 127
.....	Means	3.43	8.57	7.90	7.8	.....
		DRY 1 : 4. 33% MORTAR.				
9.....	Genesee.....	4.00	12.19	11.79	.....	2 059
32.....	Wayland	4.24	12.86	11.66	.....	1 810
61.....	Ironclad.....	4.24	12.85	11.90	.....	1 412
79.....	Empire.....	4.26	12.85	11.65	.....	1 666
.....	Means	4.18	12.69	11.75	7.4	.....
		PLASTIC 1 : 4. 33% MORTAR.				
14.....	Genesee	4.28	12.86	11.70	.....	2 036
37.....	Wayland.....	4.24	12.80	11.34	.....	1 600
64.....	Ironclad.....	4.24	12.85	11.62	.....	1 434
82.....	Empire	4.38	13.24	12.00	.....	1 523
.....	Means.....	4.28	12.94	11.66	9.9	.....
		EXCESS 1 : 4. 33% MORTAR.				
3.....	Genesee...	3.83	11.62	9.54	.....	2 416
42.....	Wayland.....	4.24	12.71	11.70	.....	1 767
67.....	Ironclad	5.00	15.00	13.90	.....	1 594
85.....	Empire.....	4.42	13.24	12.00	.....	1 488
.....	Means	4.37	13.14	11.78	10.0	.....

TABLE No. 2.—(*Concluded*).

(1)	(3)	(17)	(18)	(19)	(A)	(20)
		DRY 1 : 4. 40% MORTAR.				
21	Genesee	4.34	10.97			1 972
47	Wayland	4.24	10.71	9.92		1 671
70	Ironclad	4.12	10.46	9.70		1 652
88	Empire	4.24	10.71	9.91		1 806
	Means	4.24	10.71	9.84	8.1	
		PLASTIC 1 : 4. 40% MORTAR.				
24	Genesee	4.34	10.90	10.16		1 828
52	Wayland	4.34	10.97	10.17		1 750
73	Ironclad	4.46	11.25			1 544
91	Empire	4.26	10.71	9.93		1 726
	Means	4.35	10.96	10.09	8.0	
		EXCESS 1 : 4. 40% MORTAR.				
6	Genesee	4.25	10.63	8.31		2 132
28	Genesee	4.16	10.40			1 846
57	Wayland	4.34	10.97	10.17		1 670
76	Ironclad	4.54	11.25	10.43		1 508
94	Empire	4.34	10.97			1 472
	Means	4.33	10.84	9.64	11.0	
		DRY 1 : 5. 33% MORTAR.				
10	Genesee	4.92	14.70	14.20		1 664
33	Wayland	5.16	15.41	14.39		1 537
	Means	5.04	15.05	14.29	5.1	
		PLASTIC 1 : 5. 33% MORTAR.				
15	Genesee	5.00	15.00	13.73		1 801
38	Wayland	5.00	15.00	13.60		1 586
	Means	5.00	15.00	13.66	8.3	

TABLE No. 3.

Summary of Concrete Tests in Table No. 2.

	33%				40%				
	Number of blocks.	Col. (17) of Table No. 2.	Col. (18) of Table No. 2.	Col. (19) of Table No. 2.	Per cent. of shrinkage.	Number of blocks.	Col. (17) of Table No. 2.	Col. (18) of Table No. 2.	Col. (19) of Table No. 2.	Per cent. of shrinkage.
1 to 1 Mortar.										
Dry	3	1.57	4.74	4.30	9.3	2	1.64	4.10	3.82	6.8
Plastic	3	1.83	5.51	5.01	9.1	2	1.66	4.14	3.82	7.7
Excess	3	1.70	5.11	4.64	9.2	3	1.70	4.24	3.97	6.4
2 to 1 Mortar.										
Dry	5	2.42	7.29	6.74	7.4	6	2.44	6.12	5.89	3.8
Plastic	4	2.45	7.28	6.62	9.1	6	2.50	6.28	5.83	7.2
Excess	5	2.35	7.02	6.36	9.4	5	2.60	6.47	5.97	7.7
3 to 1 Mortar.										
Dry	5	3.15	9.49	8.78	7.5	6	3.21	8.03	7.36	8.4
Plastic	6	3.30	9.92	8.89	10.4	6	3.31	8.23	7.62	7.4
Excess	5	3.25	9.72	8.83	9.2	5	3.43	8.57	7.90	7.8
4 to 1 Mortar.										
Dry	4	4.18	12.69	11.75	7.4	4	4.24	10.71	9.84	8.1
Plastic	4	4.28	12.94	11.66	9.0	4	4.35	10.96	10.09	7.9
Excess	4	4.37	13.14	11.78	10.4	5	4.33	10.84	9.64	11.1
5 to 1 Mortar.										
Dry	2	5.04	15.05	14.29	5.1	1	4.42	11.25	.....	.....
Plastic	2	5.00	15.00	13.66	9.1	1	5.00	12.50	11.56	7.5
Excess	1	5.08	15.20	13.60	10.5	1	5.24	12.90	.....	.....

DISCUSSION.

Mr. Fowler. C. E. Fowler, M. Am. Soc. C. E.—This subject has impressed the speaker as being a very important one, for the reason that in figuring on work he has, as a rule, found that engineers and contractors know only approximately the quantities of cement, sand and broken stone to order for a given number of yards of concrete of certain specified proportions. The most convenient way of determining these quantities is to prepare tables from which the amounts required for a yard of concrete of any proportion may be taken, using only such proportions as have the voids in the stone entirely or more than filled.

Many writers on this subject have assumed that every concrete, no matter what the proportions may be, has simply a yard of broken stone, without regard to the fact that there is more or less mortar in different proportions. In making up a table of this sort the speaker used the proportions of mortar given in Baker's "Masonry Construction," but found that this did not agree with actual practice. These tables in "Masonry Construction" were made up from theoretical considerations, and it is understood that they are to be revised in a new edition.

TABLE No. 4.—Portland Cement Concrete.

No.	Proportions.	Barrels cement.	Cubic Yards sand.	Stone, 0.4 voids.	Stone, 0.5 voids.
1	1-2-3	1.77	0.51	0.87	1.05
2	1-2-3½	1.68	0.49	0.91	1.10
3	1-2-4	1.59	0.47	0.95	1.15
4	1-2-4½	1.48	0.44	1.00	1.20
5	1-2-5	1.39	0.42	1.04	1.26
6	1-2-5½	1.30	0.40	1.08	1.30
7	1-3-4	1.30	0.57	0.83	1.00
8	1-3-4½	1.22	0.54	0.89	1.06
9	1-3-5	1.16	0.52	0.92	1.11
10	1-3-5½	1.09	0.50	0.97	1.16
11	1-3-6	1.04	0.48	1.00	1.20
12	1-4-6	1.00	0.55	0.91	1.09
13	1-4-6½	0.96	0.53	0.94	1.13
14	1-4-7	0.92	0.51	0.97	1.17
15	1-4-7½	0.88	0.49	1.00	1.21
16	1-4-8	0.83	0.47	1.03	1.25

About 34 cu. ft. loose = 1 cu. yd. rammed.
Labor = ½ to 2 cu. yds. per man per day.

Table No. 4 was made up some years ago by comparison with actual cases, and has been found to give very satisfactory results in practice. The column for broken stone with 0.4 voids will represent

Mr. Fowler. closely ordinary broken limestone which chips up more than a harder stone and consequently has less voids; while the column with 0.5 voids represents trap rock, the break in which is more angular. It will be readily seen from the table for Portland cement concrete that certain proportions will not have the voids filled and should not be used. For example, with 1 to 3 mortar and with stone having 0.4 voids all the concrete given will have the voids filled, or more than filled, while with 0.5 voids only the 1-3-4 (No. 7) will have the voids filled. The table is made up on the basis of shrinkage under ramming of about 7 cu. ft., and 3.8 cu. ft. of cement to a barrel.

In a recent paper presented to the Society, one of the members stated that concrete was not a fit material for piers of railway bridges, unless a pedestal block of stone was used on which to rest the masonry plates. The speaker would refer that member to the engineers of half a dozen large western roads, who use concrete for piers, and allow the bed plates to rest directly upon the concrete. Concrete is certainly better than most stone, except granite, and probably 50% better than much of the stone commonly used.

Mr. Tillson. G. W. Tillson, M. Am. Soc. C. E.—The speaker has had some experience in the last few years in mixing concrete for street work, by hand and by machinery. For such work, the layer of concrete is comparatively thin, only 6 ins. thick, as a rule, and the mixer, if a machine is used, must be moved forward to keep out of the way of the finished work. In Brooklyn, for the last two or three years, almost all the concrete for street pavements has been mixed by machinery such as described by Mr. Gould. The method of procedure, however, is a little different. The sand and cement are first thoroughly mixed dry, in the proper proportions, on one side of the machine. The stone is all delivered on the opposite side and is dumped on a plank platform. Before the stone is used, it is thoroughly wet. A gas pipe, perforated throughout its length and running lengthwise inside the cylinder of the machine, admits the water. Then, when the cement and sand are so mixed as to produce a good mortar, an inspection of the material as it comes from the machine indicates whether or not the stone is in the proper proportion to give a good concrete. Good concrete cannot be obtained without good mortar. The specifications call for a 1: 2: 4 mixture, and, generally, there are two men shoveling in the material on the cement side and four men on the stone side, and the shoveling is continuous, on both sides, until the machine has to be moved. The mixer has a gate at the lower end, attended by a man who dumps the concrete into the wheel-barrows. In that way, with proper care, the result is as good as if the work was done by hand, if not better. In both cases the quality depends entirely upon the work being done properly and systematically. Good concrete can be obtained when mixed either by hand or by machinery. When

mixed by hand, the difficulty is, as has been said, to have the mixing done thoroughly. It is hard work, and in warm weather the laborers will get through with a batch as soon as possible. Mr. Tillson.

In making machine-mixed concrete the difficulty is to supply the ingredients in the proper proportions, so that there shall not be an excess of any single material. The stone and cement should be put in at regular intervals. Their supply will be graded very nicely after the men have worked at it a little while, especially if the foreman understands the work. There is great danger, however, in using a machine, if both the foreman and the men are inexperienced.

The relative costs of the two methods, as nearly as the speaker has been able to estimate roughly, by counting the number of men on gangs working by each, are about 7 cents for the machine-mixed and 10 cents for the hand-mixed concrete per square yard, 6 ins. deep. This will doubtless vary 10%, either way, according to the contractor or the foreman.

The speaker has found the amount of water that should be used to be dependent upon conditions. A dry concrete will give good results in cool weather, and the same mixture, in warm weather, will be of practically no value. When the weather is very hot, and when concrete has to be exposed to the air, as when used as foundation for an asphalt pavement, unless it is mixed quite wet the water dries out so rapidly, for the first inch at least, that there is no chance for the chemical action to set in. The result is, that the concrete is dry, and, in many cases, is readily disintegrated near the surface, when, under other conditions, good results would have been obtained. The speaker believes that it is safer to make a concrete a little wet rather than a little dry, and he had the idea that hand-made concrete was better, in the abstract, than machine-made concrete, but judging from some hand-made work seen recently, he believes it fair to say that the machine-made concrete is just as good as the hand-made, and that its quality, in both cases, depends greatly on the care taken in mixing.

H. F. Dunham, M. Am. Soc. C. E.—The presence of dust in the concrete has been mentioned. In certain parts of the country large quantities of limestone are crushed for use in blast furnaces, and in some establishments the limestone is also crushed for concrete. They have, altogether, a heterogeneous product; some of it is very coarse and is used in mills, some is of fine grades and is used for walks when it can be disposed of for that purpose, some is crushed to pass through a $2\frac{1}{2}$ or 2-in. ring, as the case may be, for regular concrete, and there is also a product, called dust-screenings, which resembles a mixture of half flour and half Indian meal, with some pieces as large as a kernel of corn. Mr. Dunham.

The speaker was desirous to see the effect of treating that dust in

Mr. Dunham. connection with concrete just as one would treat sand. He, therefore, made a mixture in proportions of 1 part cement, 2½ parts dust and 3½ parts broken stone of about the size which would pass through a 2-in. ring. A block of about 9 ins. in each dimension was made, kept for a period of two or three weeks and tested. It was loaded to the limit of 3 tons to the square foot, which was more than it was required to bear in the place where it was to be used, and showed no sign of fracture or disintegration. It seemed to be entirely satisfactory. American cement of ordinary grade was used, and, to appreciate the condition fully, it would be necessary to see the so-called dust. It was material which could not well be shoveled if there was any wind.

In an experiment regarding the voids in crushed limestone, an ordinary metal receptacle was first filled with the large broken stone, 2-in. ballast, and then with water, and it was found that the quantity of water required was substantially the same as that required to fill to overflowing the same vessel after it had been completely filled with this so-called dust; showing that the limestone, whatever its condition —probably it would hold true for any grade between those two— would require substantially the same amount of water to completely saturate it and fill the voids.

Mr. Gould. E. Sherman Gould, M. Am. Soc. C. E.—In regard to the comparative merits and quality of hand-made and machine-made concrete, the speaker thinks the impression is somewhat general that the hand-made concrete is the better, but he does not know how that has been ascertained, whether by actual examination or by thinking that it ought to be the best because it is hand-made. Hand-made work of all kinds is often looked upon as being superior. The speaker has been led to take a different view, and in his own experience, as far as he has been able to judge, without applying any critical test, such as testing the tensile or crushing strength of large blocks, but just from the appearance of the concrete, he has been led to think that the machine-made product was the better; and it is natural that it should be, because the mixing of the ingredients of concrete, being one of the hardest kinds of work that a man can do, is often slighted. The material must be turned over thoroughly; the shovels must go down to the bottom of the heap until they scrape against the boards of the platform, and the mixture must be picked up from the bottom and turned over again and again. On the other hand, when the product is put in a machine and stirred around mechanically there must be a more complete and regular mixing than could otherwise be obtained. No doubt there are several ways of making machine-made concrete. The only machine the speaker has used is one with a metallic shaft running through the box, with fins or fans working in the material; the box being on a slant and the material fed in at one end and out at the other. The comparisons of hand-made and machine-made concrete

were made on work which the speaker was executing, and when, in order to gain time, he supplemented the machine by gangs of men making the concrete by hand in the ordinary way, and using the same proportions as in the machine; the relative merits of the two products were judged in that way. The method of making the machine-made concrete was as follows: The sand and cement were measured out on the platform surrounding the machine and mixed dry, only one batch being mixed at a time. The proportions were 1 of cement, 3 of sand and 6 of broken stone. The stone was crushed in a machine. At first it was screened, but was afterward used just as it came from the crusher, and it was believed that the results were better, or at least as good. A barrel of cement was mixed dry with 3 barrels of sand, and each barrel of stone was thoroughly wet. The stone was emptied out on the platform, and then the stone and mortar were shoveled into the mixer. After a short time the men got accustomed to the rate at which the stone and mortar should go in, so that by the time all the stone was gone, all the mortar was also used up and the bed was cleaned off. The water was fed in and was regulated by a valve managed by a man who stood at the lower end of the machine so that he could see the concrete as it fed out into the iron wagons, by which it was conveyed to the work. After a little while the man who was managing the water valve knew the proper consistency; if the concrete was too wet or too dry, word was sent to him and he regulated the water accordingly. The speaker feels quite sure that no hand-made concrete, using the same proportions, could equal that machine-made product, because of the thorough way in which it was churned up. Mr. Gould.

Another point, with which the speaker agrees fully, is that although in a laboratory experiment the dry mortar or concrete is stronger, yet in practice it is very apt to be weaker, and although too much water is bad, too little is a great deal worse. That has always been an axiom with the speaker, and he has found it was difficult to get the necessary amount of tamping done to bring up the moist surface on the dry concrete, without which it is worthless and ought to be removed. If the concrete is so dry that tamping fails to bring the moisture to the surface—indeed it ought to be almost a water surface rather than mere dampness—the concrete is worthless and should be taken out. By putting in a little more water, however, this is avoided, the work is certainly done more quickly and, on the whole, a better result is obtained. The speaker was brought up in the dry school, and got his first education in concrete making from General Gillmore on work which he was executing for him in the rebuilding of Forts Sumter and Moultrie, in Charleston Harbor. All the work there was done according to General Gillmore's directions, under rules which he had given. It is certain that a very fine concrete was produced,

Mr. Gould. but it was with great labor, and was done by day's work. If anything was at all wrong the concrete was simply taken out and the work started over again, nothing which was at all imperfect being allowed to remain. The speaker, however, gradually came around to a freer use of water, and thinks that it gives, upon the whole, a better result. One reason is that in making successive beds of concrete, if they are being brought up in 4 or 6-in. layers, which is apt to be the case, the last bed tamped makes a better bond with the next bed put on if it is a little more moist. The speaker is quite convinced that a section through a large mass of concrete which has been constructed of very dry 6-in. layers would be very apt to show partings between the successive beds. That may or may not be, but the speaker would rather expect to find it, and, more than that, he would be afraid that the dry bed had immediately taken a set and that the placing of the other bed upon it and the tamping—the influence of tamping, of course, is felt through the entire bed—would have a rather disastrous effect upon the partially set concrete in the bed below; whereas, if it is more moist the set does not take place so rapidly, and there is not the same danger in pounding on a bed of concrete which has just commenced to take a set. In other words, if every possible precaution be taken and the work is done under able and constant inspection, it is probable that a little stronger and better product may be obtained with a very dry mixture; yet, taking contract work as it is done all over the country, there is a better chance to get a good result if everything is kept a little moist.

After concrete has been placed, a very important detail is to keep it constantly wet until it is covered over with whatever material is to come on it. Concrete is largely used for foundations upon which stone work is built, but in the speaker's practice all exposed concrete is kept wet for an indefinite period. He has had concrete watered every day and kept constantly wet for probably two or three weeks after it had set. In using hydraulic cement, which is the only kind ever used now, the great point is to let it always have plenty of water, and, after the concrete is all finished, no matter how much water has been used in making and placing it, it should be kept constantly wet, not by dashing water on it, but by sprinkling. The longer this can be done, the better.

The experiment mentioned by Mr. Conrow was conclusive as regards that particular hand-made and machine-made concrete, but the speaker does not think that the two can be spoken of so generally as that. Good hand-made concrete would be better than bad machine-made concrete and *vice versa*. The only way to settle the matter satisfactorily would be to test the very best of each kind.

The greater adhesion in the washed bricks was probably due to the saturation with water. If two bricks are washed and thoroughly

saturated with water their adhesion is very much greater than that of two dry, unwashed bricks. The speaker would attribute the increased adhesion to the moisture. Mr. Gould.

P. W. Henry, Assoc. M. Am. Soc. C. E.—About a month ago the speaker saw in New Orleans the work which is being done for the Drainage Commission, of which Major Harrod is Chief Engineer. The machine used in mixing the concrete is the ordinary asphalt mixer. It consists of a chamber in which two shafts provided with teeth revolve in opposite directions and draw the mixture toward the center. The sand, cement and shells are placed in receptacles above, which hold the required amount. The dry materials are dumped directly into the mixer, and are mixed dry until all are practically of one color. The water is then turned on. The speaker thinks that the quantity of water is judged by the eye and not measured, but it could be arranged to admit water by volume. The result is an absolutely perfect mixture, if such a thing is possible; because a great number of analyses of asphalt mixtures with that type of machine have shown an identical grading of the sand throughout all parts of the mixture, and also the same amount of bitumen in the mixture. The results in New Orleans are certainly very satisfactory. Whether this style of mixer would be as suitable for trap rock, for instance, which is very heavy, or even for limestone, is a question upon which the speaker is not able to express an opinion. It certainly works very satisfactorily with the shells. Mr. Henry.

The speaker's experience with dry and wet concrete has been confined entirely to laying concrete for asphalt pavements, and indicates that it is safer to use a moderately wet mixture for that purpose. When concrete is laid during hot weather, it is a very good practice to sprinkle the surface every night until the concrete is thoroughly set, or for, say, eight or ten days.

Herman Conrow, Jun. Am. Soc. C. E.—In ten of the tests given by the author for "dry," "plastic" and "excess" concrete, under the same conditions, the speaker finds that the average ratios of strength are as follows: Mr. Conrow.

Dry, 29.1; plastic, 26.6; excess, 25.3.

About three years ago the speaker made some experiments upon concrete with especial reference to its economical mixing and laying. The experiments were made under average conditions of mixing by hand, and the figures here given were obtained with a gang of ten men and a foreman. When laying wet concrete, the gang was divided as follows: One foreman, 9 mixers, 1 rammer. An average day's work was 15 cu. yds. of concrete in place. When laying dry concrete, the gang was divided thus: One foreman, 6 mixers, 4 rammers; and, under the same conditions as above, an average day's work was 8 cu. yds. in place. Taking the foreman's wages at $2 per day, and the men at $1.50 each, the cost of laying 1 cu. yd. under the two conditions was as follows:

Mr. Conrow.

Cost of Laying Wet Concrete per Cubic Yard.		Cost of Laying Dry Concrete per Cubic Yard.	
Cement (1 barrel Portland)	$1.50	Cement (1 barrel Portland)	$1.50
Sand	.50	Sand	.50
Stone	1.00	Stone	1.00
Labor	1.13	Labor	2.12
Total	$4.13	Total	$5.12

The dry concrete thus cost 24% more than the wet. From the averages of the previously mentioned ten tests, given by the author, dry concrete is about 15% stronger than wet. Consequently an engineer would not be justified in using dry concrete except in cases where unusual strength was desired, as in arches.

Only those who have handled concrete under actual conditions can realize the greater ease of mixing and laying it when water is freely used. Especially is this true in hot weather. Dry concrete may be an uncertain material, and if the inspector relaxes his vigilance for a few minutes a weak spot is likely to occur in the masonry. Although dry concrete, by a great amount of ramming, may be made stronger than wet concrete, yet if it is not thoroughly rammed it may possess almost no strength, even when the cement is first class. Wet concrete cannot be weak—provided the cement is good and it is reasonably well mixed.

The speaker also experimented with different methods of mixing concrete. After trying various ways the following proved to be the best. Mix the mortar dry; then wet it and mix until all is of the same consistency and about as wet as can be handled easily with shovels. After the stones have been spread evenly to a depth of about 8 ins. and washed clean, spread the mortar over them and then begin the turning. By using wet mortar it saves at least one turning as the mortar percolates through the stones; and three turnings with wet mortar will incorporate more thoroughly than four turnings with dry mortar.

As to the comparative merits of hand-mixed and machine-mixed concrete, the speaker prefers the former, because in most machines there is given no opportunity of thoroughly washing the stone before it is mixed with mortar. The dust rising continually in the machine settles on the stone prior to contact with the mortar, all to the detriment of the concrete. Only by mixing in the open air with plenty of room can the stone be kept perfectly clean until the mortar is applied. The importance of thoroughly mixing the concrete is only equalled by that of having the stones clean.

There is no material used in engineering which is so variable as concrete, and the remedy for a vast amount of poor concrete is the same as in a majority of cases of poor brickwork and weak masonry—the use of plenty of water during construction.

In tearing out some concrete, the speaker noticed that when the

hand-mixed concrete was broken with a sledge hammer, it would almost always break across the stone, thus showing that the concrete was about as good as it was possible to make it. With the machine-mixed concrete, however, it often happened that the stone did not break, but broke away from the cement and left a cavity, showing, evidently, that there was weakness at that point. It is well known that if the stone is covered with a film of dust, the concrete will not be so strong as if it is perfectly clean. The speaker does not see how it would be possible, in the machine described by Mr. Henry, where all the dry materials are first mixed together and then wet, to get a perfect concrete. If, for instance, two bricks, as they come from the brickyard, are laid with a layer of mortar between them, and two other bricks, first washed perfectly clean, are similarly laid, the difference in tensile strength between the two would be very great. The same principle applies to concrete. If all the ingredients are mixed together dry in a machine, the case is exactly the same; dust-covered stone is used and the resulting concrete is not as strong as concrete made from thoroughly washed stone. Mr. Conrow.

Allen Hazen, Assoc. M. Am. Soc. C. E.—The proposition advanced by the author that the best method of determining the proportions of concrete is, first, to decide upon the ratio of sand to cement in the mortar, and, second, to determine the ratio of mortar to stone, is undoubtedly correct, although the conventional method of statement of the relative volumes of the various ingredients is extremely convenient and will doubtless continue to be used. Mr. Hazen.

The paper treats the subject apparently only from the standpoint of the crushing strength of the concrete produced. In very large masses and in foundations the crushing strength is, perhaps, the most important quality of concrete; but concrete is used for many purposes where other properties than crushing strength are of very great and, in fact, of controlling importance. Thus, in exposed positions, a concrete which will resist the action of frost is of the first importance. In other cases, a water-tight concrete is required, and in still others, it is desirable to have concrete which will completely fill forms and yield smooth surfaces, and is of such consistency as to allow the upper surface to be troweled. For all of these purposes a concrete as free from voids as possible is desirable; and to secure satisfactory results it is necessary to use an amount of mortar somewhat more than sufficient to completely fill the voids in the stone.

The proportions of voids in broken stone, gravel and sand are often very large. They are frequently underestimated by defective methods of measurement. The most obvious, and, perhaps, the most common, method used to determine voids is to place the material in a receptacle of known capacity, and to pour as much water into it as possible, measuring the volume of water used. This procedure invari-

Mr. Hazen. ably gives results which are too low, the amount of error depending upon several conditions. The sand or gravel for examination is rarely perfectly dry. Two or 3% of water by weight do not make a material appear particularly moist; but 2 or 3% of water by weight are equal to from 3 to 5% by volume, and if the voids are measured by displacement the results will be too low by this amount.

Another and oftentimes a more serious source of error arises from the fact that all the air is not displaced by the water. With coarse, clean gravels the air can be displaced almost completely, and the error from this source is not large. With dirty materials the error is much greater, and with sands the air cannot be even approximately displaced in this way.

The most accurate and convenient method of determining the voids, known to the writer, is to determine, first, the specific gravity of the solid particles, and, second, the specific gravity of the material as a whole, including the voids. The percentage of space occupied by the particles is then obtained by dividing the specific gravity of the mass as a whole, including the voids, by the specific gravity of the solid particles, and the voids are represented by the difference between the whole volume and the space occupied by the solid particles.

In case the material used for this purpose is not absolutely free from water, the whole or a part of it must be dried after the experiment to determine the amount of water in it, and a correction applied therefor. It will not do to dry a moist material before taking its specific gravity, as the closeness of packing will be very likely to be different for wet and dry materials.

It is usually a comparatively easy matter to determine the specific gravity of the solid particles, but for large classes of materials the specific gravities are so uniform that the errors introduced by assuming an average value of 2.65 are very slight. The specific gravities of sands and gravels of the glacial drift of the Eastern States and of all silicious sands are very close to this figure, and much of the broken stone used for concrete is of nearly the same density. The proportion of voids is thus a direct function of the weight per cubic foot, after allowance is made for moisture. Loosely placed stone, or sand weighing 83 lbs. per cubic foot, has 50% of voids. When compacted to 45% of voids, the weight is increased to 91 lbs. per cubic foot. In the speaker's experience this represents approximately the condition of ordinary building sand as it is usually measured for concrete, and corresponds closely with the weight of sand mentioned by the author. When further compacted to 40% of voids the weight is 99 lbs. per cubic foot. With 35% of voids the weight is 108 lbs., and with 30% of voids, 116 pounds per cubic foot.

The voids in clean gravels or in broken stone without special packing are seldom less than 40%, although they are less when sand,

small fragments of stone or other fine materials are present in considerable amount in the interstices of the larger material. When ballast for concrete contains such material, the amount of sand in the mixture should be reduced by a corresponding amount, but no reduction should be made in the quantity of cement. That is, the fine material in the ballast should be computed as sand, and as entering into the composition of the mortar and not as a part of the ballast. Mr. Hazen.

The proportions of mixing used by the author, namely, with mortar equal to 33 and 40% of the volume of the broken stone, would seem likely to result in the production of concrete with a large proportion of voids. The speaker has never had occasion to use a broken stone which would yield a concrete entirely free from voids when used with as little as 40% of its volume of mortar, and he would expect with as low as 33% that the voids would be large and conspicuous. That this was the case in the author's experiments is suggested by the fact that the percentage of shrinkage on ramming, given in Table No. 3, is substantially the same for 33% of mortar and for 40% of mortar. If the 33% of mortar had filled the voids there would have been an excess of mortar when 40% was used, and the shrinkage by ramming would have been less. As the shrinkage was as much with 40% of mortar as it was with 33%, it seems almost certain that with the smaller quantity the voids were not entirely filled; and in view of general experience, the speaker would be inclined to question whether even the 40% was adequate to fill all the voids.

There are three possible conditions for the amount of mortar in concrete: First, the mortar may be insufficient to fill the voids between the stones; second, it may just fill them; or, third, there may be an excess of mortar. In the first condition, with the voids not entirely filled with mortar, the stones will come to a direct bearing upon each other, and if there is sufficient mortar to prevent them from slipping upon each other, the crushing strength of the concrete may be as great as if more mortar had been used; and for certain purposes such concrete is entirely satisfactory. If the mortar exactly fills the voids the strength certainly will not be less, and for many purposes the concrete will be more satisfactory. Owing to the impossibility of securing perfect mixing, it is practically necessary to use more than enough mortar to fill the voids, in order to make certain that the voids will be filled at every point; and this is the reason usually given for the use of more mortar than the equivalent of the voids, but it is also possible that for some purposes an excess of mortar in itself is desirable.

The use of a sufficient amount of cement to fill the voids in the sand is probably desirable for some purposes, but it is often less necessary than the filling of the voids in the gravel with mortar. Thus, a mortar composed of one part of Portland cement and three

Mr. Hazen. parts of sand is very strong and durable, and sufficient for many purposes, although it can hardly be considered that this amount of cement will entirely fill the voids in the sand. The remaining voids, however, are so small that the mortar is nearly impervious to water, and concrete made from it takes substantially as smooth a finish as that made from a richer mortar.

It should be remembered also that it is not possible to make a concrete entirely free from voids. This arises from the fact that concrete, to be handled conveniently and satisfactorily, requires to have a certain portion of water in it. Some of this water combines with the cement in the process of setting. In some experiments made by the speaker the proportion of water combining in this way amounts to about 8% of the weight of the cement, or to about 1% of the weight of the concrete. This quantity of water is entirely inadequate to give the concrete the proper consistency for the purpose of placing and ramming. In some experiments made by the speaker the proportion of water in concrete as used in actual work amounted 3 or 4% of the weight of the concrete as placed, in addition to the amount required for combination with the cement. To mix the concrete much dryer than this requires a great deal more ramming, which is apt to be neglected under other than very severe inspection, and it is impossible to get smooth, finished surfaces with concrete which is too dry. The above-mentioned percentage of water amounts to 8 or 10% by volume, exclusive of the water which goes into combination with the cement. After the concrete has set, this water will evaporate in the course of time, if it has an opportunity, and the resulting concrete will have a corresponding volume of voids. With the method of manufacture commonly used it is not easy to see how these voids can be entirely avoided, but it is perfectly practicable to have all the voids of extremely small size and in the mortar, and to have no voids visible to the eye and resulting from an insufficiency of mortar in the ballast. Practically, the best concrete, under ordinary conditions of mixing and ramming, and with gravel or broken stone having a specific gravity of about 2.65, will rarely weigh more than 150 lbs. per cubic foot, which corresponds to 9% of voids. Concrete of this compactness, however, with proper materials and well set, resists very well the action of the weather, and the voids are not usually apparent to the eye, or sufficient to allow water to percolate through them to an important extent.

There is a matter in connection with the proportions of mixing concrete which perhaps gives rise to even more perplexity than those mentioned by the author, namely, the measurement of the volume of the cement. The volume of a barrel of cement may be taken as that of a barrel when full, or as the actual volume of the cement in the barrel after it has been shaken down by transportation to a somewhat smaller volume, or the cement may be turned out and measured in

Mr. Hazen.

another receptacle. The speaker considers that the ideal condition is to mix concrete by weight, and, as an approach to this condition, in his own practice he has often made the arbitrary assumption that a barrel of cement weighing 380 lbs. net, occupies a space of 3.8 cu. ft., making the weight of cement 100 lbs. per cubic foot. The weight of the cement is determined or checked, no attention is paid to its actual volume, but for the purpose of computation one cubic foot is assumed for each 100 lbs. of cement. The average weight of broken stone or gravel is also approximately 100 lbs. per cubic foot, and on this basis the volumes also express the weights of these materials. The sand is usually lighter, but after the actual weight per cubic foot is determined, the approximate weight can be determined from the volume.

In computing the amounts of the various materials required to make a cubic yard of concrete the following procedure has been used by the speaker with satisfactory results. The weight of a barrel of cement is taken, and 8% is added for water of combination. To this is added the weight of sand free from water which will be used with a barrel of cement, and the corresponding weight of ballast. An estimate is made of the weight per cubic foot of the finished concrete, which, for compact work, may be taken at 150 lbs. per cubic foot, and for concrete with more ballast in proportion to sand, at a lower figure, which can be determined by experiment or estimated approximately from the appearance of the finished work. The ratio of the sum of the weights of these materials to the assumed or determined weight of a cubic foot of finished concrete will give the number of cubic feet made from a barrel of cement. The quantities of the various materials required to make a cubic yard of concrete can then be readily computed. Thus, in a recent case, concrete was mixed in the proportions of one part of cement, computed as above, three parts of sand weighing 90 lbs. per cubic foot, and five parts of broken stone and gravel weighing 100 lbs. per cubic foot. The concrete when dry weighed about 147 lbs. per cubic foot. The proportions of mixing differed slightly in different parts of the work, but the above proportions are about the average.

The computed weight of concrete made from a barrel of cement was as follows:

One barrel of cement	380 lbs.
Water of combination, 8%	30 "
Sand, 11.4 (3.8 x 3) cu. ft., at 90 lbs., dry	1 026 "
Gravel, 19 (3.8 x 5) cu. ft., at 100 lbs., dry	1 900 "
Total weight of concrete made from 1 barrel of cement	3 336 "

Volume produced, $3\,336 \div 147 = 22.7$ cu. ft.

Cement required for 1 cu. yd., $27 \div 22.7 = 1.19$ barrels.

Mr. Hazen. From the records of the work 15 085 cu. yds. of concrete were made from 18 584 barrels of cement, or 1.23 barrels of cement per cubic yard, no deduction being made for ordinary waste, nor for concrete outside of contract lines.

This concrete contains very few voids visible to the eye, when broken or as exposed where it has been placed against forms. All top surfaces were troweled smooth before setting and the proportions were such as to allow this to be satisfactorily done. The proportion of sand to gravel necessary to secure this result was determined by experiment, and was the lowest which could be satisfactorily used. With the volume of sand equal to half of that of the ballast, there were too many voids exposed when the forms were taken down, and there was difficulty in making the top surfaces smooth by troweling. It was thus necessary to use a volume of sand equal to 60% of the volume of the ballast, but for other purposes a wider mixture would be satisfactory and cheaper.

Mr. Harrod. B. M. Harrod, Past-President, Am. Soc. C. E. (by letter).—This paper is very practical and useful. The tables give exact information concerning the losses in bulk of the several ingredients when made into concrete, with which those who have used that material have been familiar in a general way. Five units of cement give about four units of mortar. This is all absorbed in the voids of the stone, which also loses about 10% of its bulk by the readjustment of its particles under the rammer.

In the writer's note-book there is a very old memorandum, to the following effect:

"The theory of concrete-making is to take a barrel of shells, and pour in as much water as the interstices will admit. Then put in a quantity of mortar equal to the bulk of water admitted. If you make any deviation from this, it will be by increasing the mortar."

The authority for this formula is the late General Joseph G. Totten, Chief of Engineers, U. S. A., Hon. M. Am. Soc. C. E.

Adopting this theory, good results can always be assured, whenever there is any doubt concerning the proportion of voids in the stone, as there often is, by specifying a test, with a tight box, containing about half a cubic yard. When this is filled with loose stone, sufficient water should be poured in to fill the voids. After the stone and the sides of the box are saturated and the water is withdrawn, a second charge of water should be poured in and carefully measured. The mass of mortar used should exceed that of the water by 5 or 10 per cent.

The shells alluded to in General Totten's formula are largely used in New Orleans in the place of broken stone on account of their fitness and cheapness. They cost from $1 to $1.25 per cubic yard, which is about half the cost of good broken stone or clean gravel. They are the shells of a small clam (*gnathodon cuneatus*) about the size of a half

dollar. They are found in banks, washed up along the shores of the brackish bayous and lakes in the vicinity of New Orleans, and also in kitchen middens, in such quantities that they are dredged up free from any impurity except a little sand. They have been largely used for roads for many years, and the end of the supply is not in sight. They make a solid concrete, as the thickness and material of the shell give it ample strength, and its concavity forms a good bond. They require a little more mortar than broken stone, and the mixing must be very thoroughly done. The weight of shell concrete is 137.8 lbs. per cubic foot, which is less than that of concrete made with broken stone. This fact, in some cases, is of importance. Mr. Harrod.

WILLIAM M. HALL, M. Am. Soc. C. E. (by letter).—The writer readily admits that the new nomenclature which Mr. Rafter suggests is very good, but can see no reason why the old nomenclature should obscure from those who are looking for it the true theory of concrete mixtures. Mr. Hall.

Referring to the amount of mortar required for mixing with the aggregate, the author well says, "there being, indeed, very little necessary except to insure the filling of the voids in the aggregate material with mortar." It appears well, also, to emphasize the fact that the same statement is true in reference to filling the voids in the sand with cement; also, in reference to the volume of the voids in ordinary sands and aggregates; and, also, that where the sand, cement and aggregate are accurately proportioned, with reference to voids, unless they are thoroughly mixed, the concrete may be, in places, almost worthless.

In lock and dam construction in Kentucky, the writer found voids in sand and aggregate as follows:

The average voids in sand was 31% of the volume.

The sand was clean and sharp, and the size of grains as follows:

Held by No. 20 sieve, 11 per cent.

Passed " 20 " and held by No. 30, 14 per cent.

" " 30 " " " 50, 53 "

" " 50 " 22 per cent.

The voids in crushed stone and in gravel and mixtures of the two were as follows:

100%	of crushed	2½-in.	stone;	voids,				48%	of volume.
80%	"	2½	"	"	20%	1½-in.	gravel, voids	44%	"
70%	"	2½	"	"	30%	1½	" " "	41%	"
60%	"	2½	"	"	40%	1½	" " "	38½%	"
50%	"	2½	"	"	50%	1½	" " "	36%	"
					100%	1½	" " "	35%	"

The above results are the averages of a number of tests made from several different barge-loads of material. However, the volume of voids in the various mixtures ran quite uniform. The sand and gravel were very clean Ohio River products, taken from the shoal at the mouth of Green River, Ky. The stone was crushed Green River blue limestone,

Mr. Hall. and in texture and color was much like the Hudson River trap. The stone passed a $2\frac{1}{2}$-in. screen and also a fine one to clean it of dust. The gravel passed a $1\frac{1}{2}$-in. screen and then a fine one to remove the sand.

In sea-coast fortification construction in Massachusetts, during the season of 1898, the writer found the volume of voids in sand and aggregate to be as follows:

The average of voids in sand was $31\frac{2}{10}\%$ of the volume.

The voids in stone and gravel and mixtures of the two were as follows:

100% of crushed	$2\frac{1}{2}$-in. trap; voids,							50%	of volume.
60%	"	$2\frac{1}{2}$	"	"	and 40%	$1\frac{1}{2}$-in. gravel, voids		$38\frac{1}{4}\%$	"
50%	"	$2\frac{1}{2}$	"	"	" 50%	$1\frac{1}{2}$	"	"	" 36% "
					100%	$1\frac{1}{2}$	"	"	" 35% "

The stone was Hudson River trap with the dust removed. The sand and gravel were taken from Long Island Sound, near Willets Point. The gravel was clean, but the sand contained about 8% of loam.

By the foregoing, the writer wishes to call attention to the saving which may be effected by mixing gravel with stone, as an aggregate, instead of using stone alone. He has failed to find similar records in the literature on the subject; but as the voids in these two stones, and the corresponding mixtures with gravel are so nearly uniform, he thinks that nearly the same results will be found with other hard stones having lines of fracture similar to these.

The comparison which the author makes between the concretes made with dry, plastic and very wet mortars are especially interesting. They would be much more instructive, however, if they had included a series of cubes which had seasoned for three or four years.

The conclusions likely to be drawn from these tests are, that dry concrete, when set and hardened, is stronger than wet. It is well known that this result may be obtained from 1-in. mortar briquettes a few months old, and the reverse result from those several years old. Moreover, in tamping the test pieces, it may be assumed that all kinds of concrete were equally well tamped; whereas, in actual practice, the obtaining of well-tamped work with dry concrete is much more difficult than with plastic or wet concrete. In actual construction, the writer prefers concrete with plastic or slightly wet mortar to that which is dry. After setting has commenced, he believes it is advantageous to apply water to the outside surface, or to the interior of the concrete mass by means of well-holes.

Mr. Baker. Ira O. Baker, M. Am. Soc. C. E. (by letter).—The author deserves credit for the thorough investigation given the practical problem presented, and the special thanks of the profession for the full and careful presentation of the data, particularly the valuable summaries and comparisons.

The writer is much interested in the effect of the amount of

mortar upon the strength of the concrete. Half of the specimens tested contained mortar equal to 33% of the broken stone, and half an amount equal to 40 per cent. Apparently, the broken stone contained 43.3% of voids before tamping and 37.4% after tamping,* and the mortar was measured without tamping. Therefore, owing to the decrease of the volume of the mortar in ramming the concrete, and to the effect of moisture in increasing the volume of the sand, it is probable that the 40% of mortar was not much, if any, more than equal to the voids in the broken stone, when both were rammed. Under the same assumption, the 33% of mortar was equal to about 80% of the voids in the broken stone after the concrete had been tamped into place. Mr. Baker.

Whether or not the 40% of mortar filled the voids, one would naturally expect that an addition of 25% of mortar would give, at least approximately, 25% additional strength. The results of the experiments appear not to support this conclusion.

In Table No. 5 the results are summarized according to the plasticity of the mortar, from which it appears that concrete containing 40% of mortar is only 4% stronger than that containing 33 per cent. Apparently, the plasticity of the mortar makes no material difference in the result.

TABLE No. 5.

Plasticity of mortar.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
Dry	2 408	2 532	105%
Plastic	2 259	2 329	103%
Excess	2 133	2 227	104%
Mean	2 267	2 363	104%

Summarizing the results according to the proportions of the mortar gives Table No. 6, from which it appears that the concrete containing 40% of mortar is only 3% stronger than that containing 33 per cent.

TABLE No. 6.

Proportions of the mortar.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
1 : 2	2 640	2 820	107%
1 : 3	1 893	1 905	102%
1 : 4	1 684	1 689	100%
Mean	2 072	2 138	103%

* Report of the New York State Engineer and Surveyor for the year ending September 30th, 1894.

Mr. Baker. The mean increase in strength, from Tables Nos. 5 and 6, differs; owing partly to omitted decimals and partly because the two tables do not refer to exactly the same experiments. The writer, to save labor, used the summaries in the original paper, and they were prepared for a different purpose.

The most interesting deduction from Table No. 6 is that the strength of the concrete increases with the richness of the mortar; and therefore it is better to increase the proportion of the cement in the mortar than to increase the proportion of mortar in the concrete. This is doubtless due to the greater ease with which the richer mortar will flow into the voids during ramming.

Summarizing the results according to the method of storage gives Table No. 7, from which it appears that the method of storage is unimportant. Figured in this way, the concrete with 40% of mortar is 2% stronger than that with 33% of mortar.

TABLE No. 7.

Method of storage for first four months.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
Under water	2 424	2 448	101%
In a cool cellar	1 992	2 009	101%
In an open shed	1 932	1 999	104%
Under wet burlap	2 007	2 096	104%
Mean	2 089	2 138	102%

Summarizing the results according to the kind of cement used, Table No. 8 is obtained. It is remarkable that the excess strength

TABLE No. 8.

Brand of Portland cement.	Amount of Mortar.		Strength of the 40% concrete in terms of that of the 33%.
	33%	40%	
Genesee	2 587	2 344	91%
Wayland	2 425	2 441	101%
Iron Clad	2 142	2 295	115%
Empire	1 970	1 955	107%
Champion	2 206	2 244	91%
Mean	2 266	2 256	101%

for the concrete containing 40% of mortar, differs so much for the different cements.

It is hoped that the author will explain why the greater percentage of mortar adds so little to the strength of the concrete, and also why

this increase varies so greatly for the different cements. The writer is unable to get any light on these questions from the published data. The experiments are so extensive, and appear to have been conducted so carefully, that it does not seem reasonable to assume that variations therein are chargeable with the above remarkable results. Mr. Baker.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The author's criticism on the present practice of making concrete will be concurred in by every engineer who has given the matter any study. It is a humiliating spectacle to see first-class material, manipulated in an irrational and wasteful manner, developing results which cannot be shown to possess a single redeeming feature. Mr. Le Conte.

The tabular results submitted by the author are full of interesting matter, and bring out prominently one feature of considerable practical importance which is not fully appreciated by engineers. The crushing strength of ordinary cement mortars diminishes very nearly in proportion to the quantity of sand added; whereas the author's tables show conclusively that, in the case of concrete, this general law is materially modified. The same general law of diminution of strength follows until the theoretic mortar, 3 to 1, is reached; but for some reason, in the case of all the lower grades of concrete blocks, made with 4 to 1, 5 to 1 and 6 to 1 mortars, the corresponding diminution in crushing strength is noticeable, but much less in degree than would naturally be expected.

Taken as a class, the writer thinks the lower-grade concretes are very much neglected by engineers. In most cases where concrete is used the unit-strength of the concrete is not called into play at all. Dead weight and the distribution of pressures are all that are required, and the lower-grade concretes fulfil these conditions completely and economically.

It is to be regretted that the author did not state clearly the age of the concrete tested, the footnote leading one to suppose that the blocks were about 12 months old.

As to the relative merits of dry, plastic and excess mortars, the writer thinks there is much more danger to be expected from the lack of water than from an excess.

H. Von Schon, Esq. (by letter).—During the spring and summer of 1898, investigations of concrete were carried on under the writer's direction at Sault Ste. Marie, Mich., for the purpose of establishing the most satisfactory and economic constituents and mixtures of concrete to be used in the substructure of a hydraulic power house. The structure in question, being partly subaqueous, must needs be of masonry, and as the only available native material, Potsdam sandstone, was considered of insufficient homogeneity, concrete was selected. The character of the construction called for materials of great density and strength, and the climatic condition for resistance to low temperature, while the extent of the work, requiring about 50 000 cu. yds. of Mr. Von Schon.

Mr. Von Schon. concrete, suggested a diligent search for the most economical mixture which would secure the desired result. It was therefore to determine the kind of cement and aggregate, and the ratio of mixtures which, together, would yield the proper material for the purpose in hand, that these tests were entered upon.

The general programme as outlined for these tests was:

First.—To operate practically, that is, to avoid the introduction of refinements which could not be followed in actual construction.

Second.—To establish the characteristics of all materials used, namely, the analysis and normals of the cements; the weight, voids, and fineness of the sand; and the weight, voids, sizes and absorption of materials to be used as aggregates.

Third.—To make two concrete bars from each of the following combinations: Four different kinds of cement, four different materials for aggregates and four different ratios of mixtures.

Fourth.—To test each bar to the breaking point, when 60 days old, by applying a load at the center.

All the manipulations were entrusted to two men who had had the advantage of several years' training in the United States cement testing laboratory at this point, under L. C. Sabin, Assoc. M. Am. Soc. C. E., U. S. Assistant Engineer, and Mr. W. W. Dann, Assistant Engineer, who had direct supervision and kept the records. A building was conveniently arranged with material bins, testing and mixing rooms. Storage for the manufactured bars was provided for outside the building.

TABLE No. 9.—Characteristics of Cements.

		Class *E.*	Class *R.*	Class *F.*	Class *N.*
Analysis.	Silica	23.08	21.70	27.48	23.16
	Alumina	5.69	8.76	.....	6 33
	Iron oxide	5.35	1.27	14.42	1.71
	Lime	62.38	63.55	53.39	36.08
	Magnesia	1.21	2.96	3.18	20.38
	Potash and soda	1.66	1.12	....	5.26
	Carbonic acid, etc	0.60	0.50	4.36	7.07
Weight per cubic foot, in pounds		103.08	98.45	77.70	62.78
Fineness passing No. 100 sieve, per cent.		91.4	90	99.00	.....
" " No. 200 " "		73.05	71	95.00	.....
Initial set		1 h. 35 m.	2 h. 25 m.	1 h. 35 m.	0 h. 30 m.
Hard set		3 h. 10 m.	4 h. 40 m.	2 h. 30 m.	1 h. 00 m.
Tensile strength 1 : 3		Lbs.	Lbs.	Lbs.	Lbs.
7 days		235	243	170	32
28 days		340	322	243	67

Four different kinds of cement, representing the chief raw materials from which American cements are manufactured (calcareous rock, calcareous earth, natural cement rock and furnace slag), and each of them a cement of high standing among its class, were used in these tests. The sand was procured from the bed of St. Mary's River,

TABLE No. 10.—Characteristics of Sand and Aggregates. Mr. Von Schon.

	Sand.	Sand-stone.	Boulder stone.	Gravel.	Furnace slag.
Size—					
Passing 1½-in. ring, per cent.	.....	100.00	100.00	100.00	.....
Retained on 1-in. ring, per cent.	.....	100.00	100.00	10.70	.....
" " ¾-in. ring, " ...	.....	.....	.....	23.65	1.10
" " No. 4 sieve, per cent.	.....	.....	.....	8.70	2.86
" " " 10 " "	.....	.....	.....	17.14	45.62
" " " 20 " "	4.17	.....	.....	21.76	36.92
" " " 30 " "	12.52	.....	.....	6.49	8.26
" " " 40 " "	44.44	.....	.....	5.96	3.24
Passing " 40 " "	38.87	.....	.....	5.59	2.00
Weight per cubic foot dry, pounds....	96.45	79.32	84.00	107.36	126.23
Weight per cubic foot wet, pounds....	113.46	82.62	86.70	.....	.....
Voids in per cent. of volume.....	41.7	45.3	48.7	34.08	43.8

TABLE No. 11.—Relative Volumes of Constituents in One Cubic Foot of Concrete.

Using Sand Stone for Aggregate.

Materials.	Mixture *A*.			*B*.			*C*.			*D*.			*E*.		
	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.	Cubic feet.	Pounds.	Ratio of volume.
Cement—															
Class *E*.....	0.19	19.5	1.00	0.21	21.5	1.00	0.23	23.7	1.00	0.25	25.9	1.00	0.19	19.5	1.0
" *R*.....	0.19	18.6	1.00	0.21	20.6	1.00	0.23	22.6	1.00	0.25	24.7	1.00	0.19	18.6	1.0
" *F*.....	0.19	14.7	1.00	0.21	15.9	1.00	0.23	17.8	1.00	0.25	19.5	1.00	0.19	14.7	1.0
" *N*.....	0.19	11.9	1.00	0.21	13.1	1.00	0.23	14.3	1.00	0.25	15.7	1.00	0.19	11.9	1.0
Lime.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.06	3.8	0.3
Sand	0.45	43.7	2.4	0.50	48.3	2.4	0.55	53.4	2.4	0.60	58.2	2.4	0.60	58.2	3.1
Water.....	0.11	7.0	0.6	0.13	8.0	0.6	0.14	9.0	0.6	0.16	10.0	0.6	0.19	12.0	1.0
Sand Stone (S. S.)....	1.0	82.6	5.3	1.0	82.6	4.8	1.00	82.6	4.4	1.00	82.6	4.0	1.00	82.6	5.3

Using Boulder Stone for Aggregate.

Cement—															
Class *E*.....	0.20	21.0	1.00	0.22	23.0	1.00	0.24	25.3	1.00	0.26	27.4	1.00	0.20	21.0	1.0
" *R*.....	0.20	20.6	1.00	0.22	22.0	1.00	0.24	24.1	1.00	0.26	26.1	1.00	0.20	20.0	1.0
" *F*.....	0.20	15.7	1.00	0.22	17.4	1.00	0.24	19.0	1.00	0.26	20.6	1.00	0.20	15.7	1.0
" *N*.....	0.20	12.7	1.00	0.22	14.0	1.00	0.24	15.4	1.00	0.26	16.7	1.00	0.20	12.7	1.0
Sand	0.49	47.0	2.45	0.54	52.0	2.45	0.59	56.6	2.45	0.64	61.4	2.46	0.64	61.4	3.0
Water.....	0.13	8.0	0.65	0.14	9.0	0.65	0.15	10.0	0.65	0.16	10.6	0.65	0.20	12.5	1.3
Lime	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	0.06	3.8	0.2
Boulder Stone (B. S.).	1.0	86.7	5.0	1.0	86.7	4.35	1.0	86.7	4.16	1.0	86.7	3.85	1.0	86.7	5.0

Lime weighs 63.75 lbs. per cubic foot.

Mr. Von Schon. and is known as Point aux Pins sand. The materials used for aggregates were of the native Potsdam sandstone, granitic boulder stone, gravel and furnace slag (see Tables Nos. 9 and 10).

The ratios used were: For the "*A*" mixture, such proportions as would just fill the voids, and for "*B*," "*C*" and "*D*" mixtures of a respective addition of 5, 10 and 15% of mortar to the "*A*" mixture. A few bars were made of mixture "*E*" having the ratio of "*D*" mixture, but substituting lime for the cement in the excess mortar quantity. Table No. 11 gives the quantities and ratios of materials for each of the above mixtures, yielding theoretically 1 cu. ft. of concrete.

Knock-down forms were constructed of thoroughly seasoned timber lined with tin, the inside dimensions being precisely 6 ins. square and about 24 ins. long. The mixing was done in batches of 1 cu. ft. of concrete, from which two forms were filled. The operations were the customary ones, namely, spreading dry sand on clean platforms, adding the cement, mixing the sand and cement dry, adding water, mixing the mortar, spreading the wet aggregate on a second platform, adding mortar to the aggregate, mixing the concrete, filling the concrete in the form in 6-in. layers and ramming.

The mixing tools used were spades and hoes. Water was used from the city supply at its natural temperature, about 50° Fahr.; the quantity was determined by the condition of the mortar, which was kept just plastic, without any water standing on or running from it. The rammers used for tamping were of oak, weighing about 8 lbs. The weight of all materials used, of the forms before filling and of the excess material after filling, was taken accurately and recorded. All bars were made during April and May, 1898, and broken at an age of 60 days, during June and July, 1898.

After filling the forms, the end of each bar was marked by its serial number, weighed and left undisturbed for from 2 to 8 days, when the form was removed and the bar placed in storage. At the age of 60 days each bar was weighed and then broken. The breaking apparatus consisted of two rigid timber frames 18 ins. apart, and of a scale platform suspended from a steel beam of triangular shape. The bar was adjusted on the frame, the scale beam with knife-edge on the center of the bar, and the platform was loaded with weights, and finally by pouring sand, until the bar broke, when all weights were taken and recorded. The fractures of the broken bars were examined, and those of exceptional appearance were photographed.

Some of the broken bars were kept for a year, during which time they were exposed to the prevailing weather of the summer and winter season, the latter being especially severe and long. They were then re-examined, weighed, placed in water of natural temperature and weighed daily until no additional absorption was observable (see Table No. 13.)

TABLE No. 12.—History of Fabricated Bars and Breaking Tests. Mr. Von Schon.

Fabricating Data.					Test Data.				
No. of bar.	Cement.	Aggregate.	Mixture.	Weight of bar.	Days in		Weight of bar before breaking.	Breaking weight.	Character of break.
					Form.	Air.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1..	*E*	S. S.	*A*	73.5	2	58	71.0	1422.5	Clean and sharp.
2..	*E*	S. S.	*A*	73.5	2	58	71.0	1380.75	" "
9..	*E*	S. S.	*B*	73.5	5	55	71.5	1798.75	" "
10..	*E*	S. S.	*B*	76.0	5	55	72.0	1671.0	" "
17..	*E*	S. S.	*C*	75.0	3	57	72.25	2302.25	" "
18..	*E*	S. S.	*C*	75.25	3	57	72.5	2179.00	" "
25..	*E*	S. S.	*D*	77.75	5	55	75.25	2034.25	" "
26..	*E*	S. S.	*D*	78.25	5	55	75.75	2567.75	" "
33..	*E*	S. S.	*E*	73.25	6	54	69.25	862.00	" "
34..	*E*	S. S.	*E*	73.25	6	54	69.75	777.25	" "
3..	*R*	S. S.	*A*	76.25	2	58	72.5	1444.25	" "
4..	*R*	S. S.	*A*	72.25	2	58	69.5	1264.00	" "
11..	*R*	S. S.	*B*	75.5	5	55	73.5	1461.00	" "
12..	*R*	S. S.	*B*	75.0	5	55	73.0	1339.75	" "
19..	*R*	S. S.	*C*	76.75	5	55	74.5	2130.00	" "
20..	*R*	S. S.	*C*	76.5	5	55	74.5	2064.5	" "
27..	*R*	S. S.	*D*	75.5	5	55	72.5	2307.00	" "
28..	*R*	S. S.	*D*	77.75	5	55	75.0	2027.75	" "
35..	*R*	S. S.	*E*	72.5	6	54	69.0	1558.5	" "
36..	*R*	S. S.	*E*	74.5	6	54	71.0	1357.0	" "
5..	*F*	S. S.	*A*	72.25	5	55	69.75	621.75	Rough and crumbly.
6..	*F*	S. S.	*A*	74.5	5	55	71.5	511.5	" "
13..	*F*	S. S.	*B*	75.25	3	57	72.5	903.75	Clean.
14..	*F*	S. S.	*B*	75.00	3	57	72.25	933.5	"
21..	*F*	S. S.	*C*	73.00	5	55	71.00	1153.00	Clean but green.
22..	*F*	S. S.	*C*	76.75	5	55	74.75	1373.00	" "
29..	*F*	S. S.	*D*	77.00	6	54	74.5	962.25	" "
30..	*F*	S. S.	*D*	77.00	6	54	74.5	919.75	" "
37..	*F*	S. S.	*E*	73.25	6	54	71.0	1013.25	" "
38..	*F*	S. S.	*E*	75.25	6	54	73.0	1034.50	" "
7..	*N*	S. S.	*A*	67.0	5	55	Broke in handling		
8..	*N*	S. S.	*A*	69.75	5	55	"	"	
15..	*N*	S. S.	*B*	73.75	8	52	69.0	285.5	Crumbled.
16..	*N*	S. S.	*B*	73.75	8	52	70.25	305.75	"
23..	*N*	S. S.	*C*	73.5	5	55	69.0	284.75	"
24..	*N*	S. S.	*C*	73.5	5	55	69.0	229.50	"
31..	*N*	S. S.	*D*	73.75	6	54	69.0	351.25	"
32..	*N*	S. S.	*D*	74.00	6	54	69.75	281.00	"
39..	*N*	S. S.	*E*	74.00	10	50	69.00	267.00	"
40..	*N*	S. S.	*E*	72.75	10	50	67.5	246.5	"
41..	*E*	B. S.	*A*	79.25	4	56	77.25	2386.00	Clean and sharp.
42..	*E*	B. S.	*A*	79.00	4	56	77.25	2829.75	" "
49..	*E*	B. S.	*B*	77.25	22	38	75.5	2391.5	" "
50..	*E*	B. S.	*B*	80.25	22	38	78.75	2861.00	" "
57..	*E*	B. S.	*C*	80.25	18	42	78.75	2849.25	" "
58..	*E*	B. S.	*C*	80.25	18	42	78.5	3120.75	" "
65..	*E*	B. S.	*D*	81.75	5	55	80.0	3961.75	" "
66..	*E*	B. S.	*D*	79.75	5	55	77.75	3200.5	" "
73..	*E*	B. S.	*E*	78.00	2	58	75.5	2480.0	" "
74..	*E*	B. S.	*E*	78.00	2	58	75.25	2800.75	" "
43..	*R*	B. S.	*A*	79.75	4	56	77.5	3119.00	" "
44..	*R*	B. S.	*A*	79.75	4	56	77.5	2436.00	" "
51..	*R*	B. S.	*B*	81.25	22	38	79.5	3121.75	" "
52..	*R*	B. S.	*B*	79.5	22	38	77.5	3386.75	" "
59..	*R*	B. S.	*C*	80.0	18	42	78.75	2988.00	" "
60..	*R*	B. S.	*C*	80.0	18	42	78.50	3282.50	" "
67..	*R*	B. S.	*D*	80.0	5	55	77.75	3289.50	" "
68..	*R*	B. S.	*D*	80.5	5	55	78.25	3004.25	" "
75..	*R*	B. S.	*E*	78.25	2	58	74.00	2499.25	" "
76..	*R*	B. S.	*E*	79.25	2	58	76.50	2653.00	" "

Mr. Von Schon.

TABLE No. 12.—(*Concluded.*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
45..	*F*	B. S.	*A*	80.00	4	56	77.5	1183.25	Clean but green.
46..	*F*	B. S.	*A*	73.00	4	56	70.75	901.00	" "
53..	*F*	B. S.	*B*	81.00	18	42	79.25	1266.75	" "
54..	*F*	B. S.	*B*	79.75	18	42	78.00	1490.25	" "
61..	*F*	B. S.	*C*	80.25	6	54	78.5	1163.75	" "
62..	*F*	B. S.	*C*	78.50	6	54	77.0	1456.50	" "
69..	*F*	B. S.	*D*	77.75	5	55	75.5	1078.00	" "
70..	*F*	B. S.	*D*	79.50	5	55	77.25	978.5	" "
77..	*F*	B. S.	*E*	78.00	4	56	75.75	956.25	" "
78..	*F*	B. S.	*E*	78.25	4	56	75.75	935.25	" "
47..	*N*	B. S.	*A*	75.75	22	38	71.5	364.25	Crumbled.
48..	*N*	B. S.	*A*	79.75	22	38	75.25	414.5	"
55..	*N*	B. S.	*B*	77.25	18	42	72.25	267.0	"
56..	*N*	B. S.	*B*	77.00	18	42	70.5	560.0	"
63..	*N*	B. S.	*C*	77.75	9	51	72.75	596.0	"
64..	*N*	B. S.	*C*	77.25	9	51	72.25	422.5	"
71..	*N*	B. S.	*D*	75.0	8	52	71.0	203.0	"
72..	*N*	B. S.	*D*	77.0	8	52	72.0	318.75	"
79..	*N*	B. S.	*E*	77.25	4	56	73.5	486.0	"
80..	*N*	B. S.	*E*	77.75	4	56	73.0	419.5	"

NOTE.—An excess of variable quantity remained over after the making of each pair of bars.

TABLE No. 13.—ABSORPTION TESTS.

No. of bar.	Age, in days.	Weight of bar.	Condition before absorption.	Weight, in Pounds, after being Submerged.	
				1 day.	2 days.
1.....	381	71	hard.	74.0	74.0
9.....	381	50	"	51.5	51.5
11.....	381	73	"	75.5	75.5
17.....	379	39	"	40.0	40.0
19.....	376	74	"	76.0	76.0
27.....	376	72	"	73.5	73.5
35.....	370	69	"	72.5	72.5
41.....	346	77	"	78.5	78.5
49.....	346	75.5	"	76.5	76.5
51.....	346	79.5	"	81.0	81.0
57.....	342	78.5	"	79.5	79.5
59.....	342	78.5	"	79.5	79.5
63.....	325	72	crumbling.	75.0	75.0
67.....	320	77.5	hard.	78.5	78.5

NOTE.—The temperature of the water in which bars were submerged was that of the water in St. Mary's River, about 50° Fahr.

The results, as observed from these tests, are summarized in Table No. 14 and Fig. 1, from which deductions can be drawn by those interested in investigations of the theory of concrete.

TABLE No. 14.—Tabulated Results. Mr. Von Schon.

Kind of Concrete.			Constant "C".	Absorption in per cent. of volume.	Kind of Concrete.			Constant "C".	Absorption in per cent. of volume.
Cement.	Aggregate.	Mixture.			Cement.	Aggregate.	Mixture.		
E.....	S. S.	*A*	9.91	9.6	*E*.....	B. S.	*A*	18.16	4.8
E.....	S. S.	*B*	12.23	7.0	*E*.....	B. S.	*B*	18.44	3.2
E.....	S. S.	*C*	15.75	6.0	*E*.....	B. S.	*C*	20.93	3.2
E.....	S. S.	*D*	18.26	.....	*E*.....	B. S.	*D*	22.64	.....
E.....	S. S.	*E*	5.87	.....	*E*.....	B. S.	*E*	18.52	.....
R.....	S. S.	*A*	9.59	.....	*R*.....	B. S.	*A*	19.49	.....
R.....	S. S.	*B*	9.91	8.0	*R*.....	B. S.	*B*	20.80	4.8
R.....	S. S.	*C*	14.76	6.4	*R*.....	B. S.	*C*	21.98	3.2
R.....	S. S.	*D*	17.32	4.8	*R*.....	B. S.	*D*	22.05	3.2
R.....	S. S.	*E*	10.31	11.2	*R*.....	B. S.	*E*	18.09	.....
F.....	S. S.	*A*	4.12	.....	*F*.....	B. S.	*A*	8.12	.....
F.....	S. S.	*B*	6.57	.....	*F*.....	B. S.	*B*	9.78	.....
F.....	S. S.	*C*	8.96	.....	*F*.....	B. S.	*C*	9.30	.....
F.....	S. S.	*D*	6.79	.....	*F*.....	B. S	*D*	8.03	.....
F.....	S. S.	*E*	7.29	.....	*F*.....	B. S.	*E*	6.84	.....
N.....	S. S.	*A*	0.00	.....	*N*.....	B. S.	*A*	2.89	.....
N.....	S. S.	*B*	2.23	.....	*N*.....	B. S.	*B*	3.05	.....
N.....	S. S.	*C*	1.87	.....	*N*.....	B. S.	*C*	3.72	9.6
N.....	S. S.	*D*	2.37	.....	*N*.....	B. S.	*D*	2.00	.....
N.....	S. S.	*E*	1.96	.....	*N*.....	B. S.	*E*	3.33	.....

Note.—Constant "C" $= \dfrac{\text{Span in feet} \times \text{Load in pounds.}}{\text{Width in inches} \times \text{Depth in inches}^2}$.

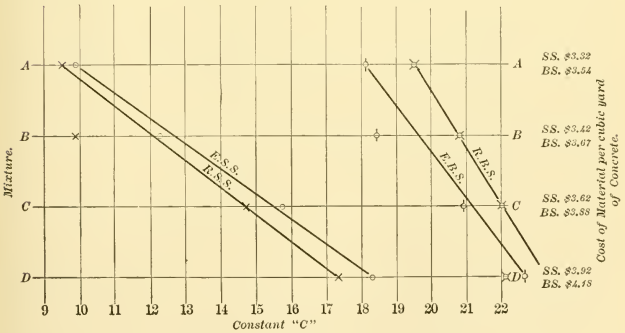

Fig. 1.

Mr. Fuller. WILLIAM B. FULLER, M. Am. Soc. C. E. (by letter).—There is one point which, it seems to the writer, has not been made sufficiently clear in this discussion, although it is really the basis of the most economical mixture for any given quantity of cement. If we make a concrete by using merely cement and sand, or cement and stone, it will take a large amount of cement to fill the voids in the sand, or in the stone, for any given quantity of concrete; but if we make a concrete of cement, sand and stone, the sand fills the larger voids in the stone and the cement has only to fill the remaining voids in the mass; a very much smaller percentage than in either of the first two cases. If, in the mixture of cement, sand and stone, the sand and stone are not in proper proportions, the cement is called upon to do more work than it should; that is, it has to fill the larger voids of this excess material. Take, for instance, sand having 45% voids, and stone having 40% voids. With the sand just filling the voids of the stone, it is easily calculated that the resultant mass has 18% voids; but supposing an excess of 10% of sand, there would be 10% of the material having 45% voids, which means there would be 2.5% more voids in the resultant mass.

The 2.5% of cement necessary to fill these extra voids, it is seen, is really wasted, for the resultant concrete would have been much stronger if the sand had been decreased and the stone increased, leaving the same amount of cement 2.5% in excess in the mass.

The above example indicates what frequently happens in mixing concrete. Indeed, it is what is frequently advocated in discussion and by writings on the subject; that is, to make up a mortar of cement and sand and mix this mortar with the stone, being sure that the mortar is slightly in excess, so as to fill all the voids of the stone.

This is what is known as drowning the stone, and the resultant concrete is hardly any better, so far as its properties are concerned, than a concrete of cement and sand alone. After the stone is once drowned no addition of mortar will improve the concrete.

The writer has frequently made a rich concrete of a poor one by simply rearranging the proportions of sand and stone without in the least changing the proportions of cement in the mass.

It must be apparent that there is just one proportion and only one in which any two such materials as sand and stone can be combined and give a minimum number of voids in the resultant mass, and it seems to the writer that this is the proportion in which they should always be mixed, whether we want a poor or a rich concrete. Mix the sand and the stone in a definite fixed quantity as determined by experiment for the particular material in use, and then add cement as economy dictates, possibly up to 10% in excess of the voids of the combined material.

This suggests another proposition, which must be almost self-evident, namely, that the proper selection and grading of the sand

and aggregate so as to produce the least number of voids will allow of a corresponding reduction in the amount of cement used, without reducing the quality of the concrete desired. In making up such mixtures, however, the necessary relation of the size of the particles of the different materials should be kept in mind, that is, the particles of each material should be small enough to enter easily the voids formed by the particles of the next larger material; otherwise, they will be left out and themselves form voids, which must be filled by the finer material or remain unfilled, to the detriment of the concrete. Mr. Fuller.

It is quite common to find sand used which is too fine for the cement, or sand in which the larger particles act merely as stone, or stone with its voids partly filled with sand or dust.

The proper proportions of sand and stone can be determined by mixing sample batches and determining their voids by any of the well-known methods; or, another very satisfactory method, as it also gives information as to the relative sizes of the materials, is to make a mechanical analysis of all the materials in the manner prescribed by the Massachusetts State Board of Health for the analysis of sands, and from the resultant curves figure the composition giving the highest uniformity coefficient. The writer has used the foregoing methods in concrete structures with uniform success in the reduction of the amount of cement required, without reducing the apparent quality, and, by changes in the proportions of the aggregates, has maintained a uniform quality of concrete, through the use of a wide range of aggregates, without changing the percentage of cement.

In discussing the relative merits of hand-mixed and machine-mixed concretes it is quite essential that the kind of machine with which the comparison is made should be stated, as there is as much difference between the products of different machines as between that of good and poor hand-mixing. With a machine of the cubical-box type, into which all the materials, including the water, can be measured accurately, the product, as far as observation can establish it, is superior to the best hand work. With such a machine the writer has seen concrete, of almost absolutely unvarying quality and consistency, produced for days at a time.

G. L. Christian, Assoc. M. Am. Soc. C. E. (by letter).—The writer has had considerable experience with hand-made concrete, but none with machine-made. Mr. Christian.

The hand-made concrete, to be of any use, requires continual, hard and conscientious work on the part of the mixers, and continual inspection by an intelligent, live and honest inspector, who will always insist on having everything done well.

The instant the inspector turns his back, the temptation to turn the water on the mortar, before the sand and cement are thoroughly mixed, and, in fact, before they are mixed at all, is too much for the

Mr. Christian. average laborer to resist, especially if he is hard pushed and the day is hot. Taking all things into consideration, the writer would expect to find machine-made concrete superior to hand-made in a large majority of cases.

Only in cases where concrete is exposed to the sun does the writer agree with Mr. Gould as to the advisability of keeping concrete moist for a long time after being placed in the work. The writer has reached this conclusion, partly as a result of a test of quite a number of briquettes which were mixed about five years ago and recently broken. Some were of neat cement, some 2 to 1 and some 3 to 1. One set of each of these was mixed with fresh water, one with 3% salt water, and the other with 13% salt water. Half of each set of briquettes was laid away on wooden shelves in a warm office soon after mixing, and the other half was placed in water and kept there until ready to be broken. Many of these briquettes, have been broken, and, so far, those of whatever kind which were kept dry have shown the greatest tensile strength.

The writer was at one time a firm believer in the dry mixing of concrete, but having seen several thousand cubic yards mixed dry and placed to a depth of 6 ins. over a reservoir bottom, and afterward over 3 000 cu. yds. mixed wet, in the proportion 1: 3: 6, and placed in the other half of the same reservoir, he became a firm advocate of wet mixing.

A careful force account was kept on the last 3 000 cu. yds., in order to ascertain the actual cost per cubic yard. The wages paid were: Foremen, $2.50; laborers, $1.35; and teams, including driver, $4.00 per day. To the cost of each item in the force account 9% has been added for superintendent, timekeeper, office help, etc., and is included in the figures here given. The cost of blasting is not included, because the contractor was paid for excavating the rock, so that the cost begins with the loading and hauling to the crusher. The average haul was 400 ft. to and from the crusher.

The cost per cubic yard is subdivided as follows:

Sand	$0.40	
Natural cement	1.18	
		$1.58
Loading and hauling to crusher	$0.27	
At crusher	0.21	
Rent of "	0.01	
Coal at "	0.05	
Hauling from "	0.16	
		0.70
Concreting gang:		
Foremen	$0.05	
Laborers	0.54	
Teams	0.09	
		0.68
Total		$2.96

ALFRED F. HARLEY, M. Am. Soc. C. E. (by letter).—The "theory of concrete" is certainly a subject upon which civil engineers should be well informed, but, nevertheless, the fact remains that the execution is invariably left to a contractor, who, while carrying out the instructions of the civil engineer, is required to guarantee the success and perfection of the work. Mr. Harley.

On one occasion the writer was consulted by the proprietor of a livery stable who had taken from a local contractor a sub-contract for hauling cement, sand and broken stone by the cubic yard, and, upon presenting the bill, was informed by the contractor that inasmuch as the concrete had required 1 part of cement, 3 parts of sand and 6 parts of broken stone, making a total of 10 parts, while the actual number of square yards of concrete laid, 6 ins. thick, was approximately 60 000 sq. yds., or about 10 000 cu. yds. of concrete, and consequently the stable proprietor could only have hauled 1 000 cu. yds. of cement, 3 000 cu. yds. of sand and 6 000 cu. yds. of broken stone. All of which argument the livery stable proprietor followed very clearly in mind, but failed to check with the actual number of loads the teams had carried, even when taking into consideration the assumption that a quantity of the material might have been dropped along the road. The writer was not made a party to the settlement finally reached, but the shrinkage of the material when rammed was certainly not taken into consideration.

The writer cannot agree with Mr. Tillson as to the economic use of a machine-mixer for street concrete work. The figures Mr. Tillson gives should at least be reversed, 7 cents being, in the writer's estimation, a minimum figure for hand-made concrete and 10 cents a minimum figure for machine-mixed concrete. Neither is the result equally good, in the writer's opinion. The wheeling for any distance in a barrow tends to bring the mortar to the bottom, while the most valuable part of the concrete, commonly called "the slush," is wasted along the run-boards and could not be picked up and used before the initial set had taken place; while the amount of water cannot be regulated as nicely in the closed trough of the machine as when allowed to run on the concrete from a hose as it is turned over by hand.

Where continuous shoveling has to be performed, as with concrete mixed by hand, it is infinitely less fatiguing to the laborer if a steel plate is used to shovel on instead of boards. Gravel is also much more easily shoveled than broken stone.

Mr. Dunham's remarks in regard to the use of broken stone and the dust obtained in breaking are interesting. In gravel deposits there is found frequently a good quality of sand in such proportion as to fill entirely the voids between the gravel. A great many civil engineers require the material to be screened, and the sand and gravel

Mr. Harley. used separately in the proportion specified. Lately, this class of material has been used by a Government official without screening, and with very good result; and it is only reasonable to suppose that at the time the gravel deposit was formed, the voids were filled entirely with the fine particles of sand washed into as close a position as they could very well occupy.

The advantage in hand-mixed concrete, in the writer's estimation, is not only in cost, but in the rapidity with which the concrete is put into position after being mixed, and the proximity of all the materials to their final resting place. One practice which the writer very firmly believes in following, is to throw a few shovels full of mortar, when mixed on the board, over the top of the last batch of concrete just previously laid, and by this means fill the interstices flush with the surface of the concrete. It might be claimed that a batch would then contain less mortar than it should, but this is not so, inasmuch as the operation is repeated over each batch thrown into position.

In New Orleans, during the execution of some contracts, the writer had an opportunity to run a hand-concrete gang at the same time that another firm of contractors was operating a concrete-mixer gang, and not only in the employment of less men, but also in the number of square yards laid during the day, were the results of the two methods under discussion apparent, and decidedly in favor of the hand-mixed concrete.

A day's work, in running three boards extending the full width of the street, would average about 900 sq. yds. of concrete 6 ins. thick, which result is all that could be wished, from the following gang:

6 wheel-barrow men—broken stone or gravel.
3 " " " —sand.
1 " " man—cement.
2 men opening cement.
7 " dry mixing.
8 " taking off boards.
3 " tamping concrete.
3 " grading concrete.
1 man attending to run planks.
3 water boys.
1 or 2 extra men, or, in all, say, 40 men.

The writer believes that Mr. Hall will find it quite customary in Europe to use broken stone, gravel, sand and cement in concrete, in certain quantities.

Mr. Rafter. George W. Rafter, M. Am. Soc. C. E. (by letter).—The author desires to express satisfaction at the appreciative discussion of the paper. That there is an important place in the literature of concrete for a paper of this character is well indicated by the remark of Mr. Fowler:

Mr. Rafter.

"Many writers on this subject have assumed that every concrete, no matter what the proportions may be, has simply a yard of broken stone, without regard to the fact that there is more or less mortar in different proportions."

The author has been greatly surprised, in looking over the literature of concrete, to see how far this fallacy has been accepted. It has also been matter of surprise to find a nearly absolute dearth of definite information as to the amount of mortar and concrete produced by definite proportions of cement, sand and broken stone, as well as the unit weight of the resultant concrete.

This paper, taken in connection with the more extended information given in the report to the State Engineer and Surveyor of New York, is an attempt to supply, in some slight degree, what appears, after study of the question, to be a very material deficiency. The results are, however, unsatisfactory to the author in this particular; that, having laid out the plan of work, no special study of the data was made until about two years after the completion of the tests. This was a mistake. The data should have been tabulated and studied from day to day during the progress of the tests. Had this been done, there seems to be no doubt that some of the irregularities of the results could have been eliminated.

The subject is extremely complicated, many apparently insignificant elements entering in to detract from the strict, scientific accuracy of the results. The more especially is this true, as in the present case, where an attempt has been made to keep the results on practical lines purely, using only such methods as could be applied in practice on work, without adding unduly to the cost of concrete. However, taking this latter condition into account, the author reiterates the view expressed in the paper, that the results are very good.

Occasion is taken at this place to especially call to the attention of those participating in the discussion, that the results of these tests are given in great detail in the annual report of the State Engineer and Surveyor of New York for 1897.* The results of a previous set of tests made in 1893 are given in the annual report of the State Engineer for 1894. Reference to all these reports should be made by anyone wishing full data.

As regards mixing concrete, either by hand or machinery, the author's experience is that with proper supervision, good concrete can be obtained by either method. As remarked by Mr. Tillson, the difficulty in hand mixing is to insure thorough work. On this point the author has found that by drilling a gang of laborers to work strictly according to formula, performing each operation in the same way, every time, there is little difficulty about securing fairly uniform results by hand mixing. Nevertheless, the author's opinion is that

*See also "Report on Tests of Metals and other Materials," Watertown Arsenal, for 1898, where may be found, also, results of other concrete tests.

Mr. Rafter. proper machine mixing is preferable, as leading to a uniformity of product which, while possible to attain by hand mixing, is still somewhat difficult. In a general way, the author agrees, therefore, with the statement of Mr. Gould, that no hand-made concrete, using the same proportions, can equal a machine-made product, because of the thorough way in which the machine churns up the entire mass.

The tests show that dry concrete is considerably stronger than that made with excess of water. As stated in the report to the State Engineer and Surveyor, the author's experience has taught him that the only way in which dry concrete can be safely used, especially in warm weather, is to wet the surface thoroughly as soon as the ramming has been completed, and to keep the surface wet until after the completion of the setting. This procedure will be likely to prevent the formation of the partings between successive beds referred to by Mr. Gould. The author, however, does not intend to be insistent as to the use of dry concrete, but rather presents the results of these tests for comment and professional criticism. He is, nevertheless, well satisfied that dry concrete, with the mortar comprising 40% of the aggregate material, can be made practically water-tight. In recent work with a concrete of this composition, a dam has been constructed, which, with one exception, shows absolutely no moisture on the face. The one exception is two pin holes, both within an area of 1 sq. ft. Their presence can only be taken to indicate careless work—for the moment, evidently, vigilance was relaxed—but this single exception can hardly be taken as proving the inapplicability of dry concrete. In any case, it is certain that plastic and excess concretes can be used much more easily than dry, and the practical question is, after all, whether the gain in strength is enough to compensate for the increased chance of poor work.

As to ultimate strength, it seems certain that time is a great eliminator of the differences between dry, plastic and excess concretes, the tendency being, undoubtedly, to show less and less difference in ultimate strength with increase of age.

The concrete blocks here discussed were made between July 10th and September 5th, 1896. They were broken between January 29th and May 10th, 1898. The ages of the blocks, therefore, varied from about 670 to about 570 days. The exact age of each block may be obtained by reference to Table No. 1 of the report to the State Engineer and Surveyor, in the annual report for 1897.

One important point which the author had in mind in making these tests was to show that, for many purposes, the formulas in common use give concretes which are unnecessarily expensive. Thus, for foundations which are entirely under water there is no object, so far as the author can see, in spending the money necessary to make an absolutely impervious concrete. For such work a 30 or 33%

concrete is apparently sufficient. The same is also true of retaining walls, for which concrete is now used extensively, and through which weep holes can be left, thus insuring that water will not penetrate any considerable distance into a 30% concrete, and which may, as the author is well aware, contain a slight percentage of voids. Mr. Rafter.

Where water-tight work is required, the mortar may be made from 40 to 45% of the aggregate. By using different sized stone for the aggregate, there seems to be no doubt that 40 to 42% of mortar will make a practically water-tight material. The author says practically water-tight, because the quality of the mortar will, in some degree, decide as to absolute water-tightness.

In the original tests of 1893, the ratio of broken stone to mortar was made mostly 42%, a few blocks having been fabricated with the ratio of 30 per cent. These blocks were all broken under the author's personal supervision, and none of the 42% blocks exhibited any voids visible to the eye. They were, in the best sense of the word, solid blocks. Blocks of 30 and 33%, however, exhibit some voids, the greatest difficulty with such concretes being to make them absolutely fill out all the voids next to the forms. This is a difficulty which is, however, very easily corrected by using a somewhat larger proportion of mortar next to the forms, as has frequently been done with good success on work.

In the series of tests made in 1896, and which are here specially considered, the author is unable to say as to whether there were any appreciable voids in the 33% concretes, but from his experience with the 30% concretes made in 1893, he would be disposed to say that the voids were very small, especially in the plastic and excess concretes. Under this head it may be pointed out that the effect of ramming is to decrease the voids. This was strongly brought out by a series of tests in which it was found that the Portage stone, broken to pass through a 2-in. ring, when slightly shaken in the measure, gave, as a mean of five trials, 43.3% voids. The same stone packed in a measure with a tamping iron, used about as forcibly as the ordinary ramming of concrete, gave, as a mean of five trials, 37.4% voids. In these tests the air was driven out of the voids entirely.

The author has little doubt that, with the lubrication of the mortar, the voids are reduced, under thorough ramming, to not much more than about 30 per cent. It is from this point of view that he decided to use 33% of mortar in the tests herein discussed, that quantity of mortar being enough to just about insure filling the interspaces of the stone. Such a concrete will not, however, be absolutely water-tight.

In the same way the excess of mortar in 40% concrete was considered to be large enough to insure practical water-tightness. It is possible, however, that slightly larger proportions than the foregoing should, in some cases, be used. On the other hand, by selecting aggregate ma-

Mr. Rafter. terials of different sizes, thus reducing the proportion of voids, it is believed that the proportions of 33 and 40% fairly meet the great majority of cases.

Mr. Conrow points out that the average ratios of strength are, for concretes with dry mortars, 29.1; plastic, 26.6; excess, 25.3; as based on the average of ten tests. From these figures he deduces that dry concrete is about 15% stronger than wet. In his experiments as to relative cost of laying wet concrete *versus* dry concrete, he found that wet concrete could be fabricated at a cost for labor of $1.13 a yard, dry concrete costing, with other conditions the same, $2.12 a yard. This is for hand labor. The author's experience is that for machine-mixed concrete, aside from ramming, the cost will be no greater for dry than for wet. As an average, the ramming of dry concrete will cost 25 cents per yard more than for wet.

Mr. Hazen's discussion of the conditions involved in the determination of voids, and as to the weight of concrete, illustrates forcibly how far we are from appreciating the practical considerations to be taken into account in making concrete, especially when the subject is approached from the theoretical point of view purely. However, as to determining the voids, the use of the method outlined by Mr. Hazen seems to the author to give results too large. Independent of its lack of applicability out on work, the author believes that the displacement method of determining voids is, for clean material, much the better. A knowledge as to the interstices in the aggregate material or ballast is wanted, and not, in any degree, the porosity of the material. The author does not wish to be understood, however, as saying that information as to the porosity of aggregate material may not be of importance, the more especially since such information is very necessary in order to decide between different samples. His point is, that, as regards concrete mixtures, what is wanted is the volume of the interspaces between the stones, independent of the porosity. Limestones, sandstones and other aggregates are often quite porous, and the method by specific gravities apparently takes into account this porosity. Limestones sometimes absorb 5% of their weight of water, and sandstones as much as 7%, while broken brick may absorb 20 per cent. As fair averages, we may take limestone at 3%, sandstone 4%, and broken brick 10 per cent. The most practical way to proceed is to first thoroughly wet the broken stone and allow it to drain before placing it in the measure. Water should then be poured in slowly and at one corner of the measure, thereby insuring the driving out of the air. For sand, absorption into the pores is so small that it may be neglected, but it is important to pour in the water slowly and at one corner of the measuring box. This procedure applies to clean sand. For sand containing much vegetable mould or other foreign matter, Mr. Hazen's remarks as to the uncertainty of the displacement method apply

forcibly. As regards the penetration of water into the pores of the aggregate material, it may be remembered that the smearing of the broken stone with mortar tends to seal up the pores in properly-made concrete, thus preventing the ingress of any considerable amount of water. In any case, taking into account the porosity of the aggregate material is a misleading refinement. Mr. Rafter.

There is no reason why, with care, any ordinarily intelligent foreman cannot, by the foregoing simple method, determine quickly the voids in aggregate material and clean sand, as often as required, and well within any limit of accuracy needed for proportioning concrete.

Reference has already been made to the original series of tests of 1893, and which are reported upon in the annual report of the State Engineer and Surveyor for 1894. The weights of the 42% blocks fabricated at that time ranged from about 145 lbs. per cubic foot to about 150 lbs., the variation in weight being in some proportion to the strength of the mortar, blocks with 1 to 1 mortar being heavier than those with 1 to 2 or 1 to 3. In some cases, cubes of 1 ft., with 1 to 1 mortar, 42%, weighed from 150 to 160 lbs. The tests of the series of 1893 are, however, unsatisfactory in this, that the exact dimensions of the blocks were not taken, and there is probably some uncertainty in the figures on this account.

In the tests of the series of 1896, and which are here specially considered, no such uncertainty exists; the exact dimensions of each block were taken carefully, and the weights per cubic foot computed therefrom. In Table No. 15 we have the mean weights of all the concrete blocks in pounds. The Portage sandstone used as ballast weighs 155 lbs. per cubic foot, and has, when crushed in 12-in. cubes, a crushing strength of 6 000 to 7 000 lbs. per square inch. A few determinations of weight of mortar used show the following mean weights per cubic foot:

			Weight per cubic foot.
Empire	cement—	1 to 2 dry	126.4 lbs.
"	"	1 " 3 "	120.9 "
"	"	1 " 4 "	119.1 "
"	"	1 " 2 plastic	124.7 "
"	"	1 " 3 "	121.0 "
"	"	1 " 4 "	119.2 "
Iron Clad	"	1 " 2 dry	128.5 "
"	"	1 " 3 "	124.8 "
"	"	1 " 4 "	120.9 "
Champion	"	1 " 1 "	126.2 "
"	"	1 " 2 "	123.8 "
"	"	1 " 3 "	119.2 "

The foregoing weights of mortar blocks, as well as concretes, included in Table No. 15, are all as taken at the time of breaking, and

Mr. Rafter. when, presumably, the excess water had evaporated. This conclusion is reached by considering that the blocks stood in a dry, warm place for some time before breaking.

Referring to Table No. 15, it appears that there was very little difference between the weights of 33 and 40% blocks. With the Genesee cement 1 to 1 mortars, the mean weights are, for 33% blocks, 146.5, and for 40% blocks, 145.6. For Wayland, 1 to 1 mortars, the mean figures are 33%, 146.9; and 40%, 145.8. For Genesee 1 to 2, the figures are, 143.8 and 143.7; and for Wayland 1 to 2, 144.5 and 144.2.

For the Genesee cement, the weight per cubic foot was taken at 99.5 lbs.; for the Wayland, 100 lbs.; for the Iron Clad, 97 lbs.; for the Empire, 99.5 lbs., and for the Champion, 87 lbs. The reason that the weights of 33 and 40% blocks are practically the same is probably because in 33% blocks the excess reduction of voids under ramming just about compensates for the extra quantity of mortar in the 40% blocks, the relation between the weights of mortar and broken stone being such as to give this result. The persistency of the figures through the whole series shows that this result is not accidental, but rather the expression of a well-defined law.

TABLE No. 15.—Mean Weights of Concrete Blocks.

(In pounds.)

		1 to 1.		1 to 2.		1 to 3.		1 to 4.		1 to 5.		1 to 6.	
		33%	40%	33%	40%	33%	40%	33%	40%	33%	40%	33%	40%
Genesee.	Dry	146.6	.....	.....	143.4	142.7	142.9	141.3	141.6	140.5	.....	139.5	.....
	Plastic	146.2	.....	143.2	143.4	143.2	142.4	142.1	140.7	141.2	.....	.....	.....
	Excess	146.6	145.6	144.5	144.4	142.9	141.9	143.1	141.2	.....	.....	.....	.....
	Mean	146.5	145.6	143.8	143.7	142.9	142.4	142.2	141.2	140.8	.....	139.5	.....
Wayland.	Dry	147.0	146.4	144.5	144.4	140.0	142.1	142.6	140.0	141.7	139.7	.....	.....
	Plastic	147.8	146.4	145.1	143.5	144.1	141.0	141.2	141.6	140.8	139.0	.....	.....
	Excess	145.9	144.7	144.0	144.6	140.7	141.6	141.3	142.2	139.8	140.4	.....	.....
	Mean	146.9	145.8	144.5	144.2	141.6	141.6	141.7	141.3	140.8	139.7	.....	.....
Iron Clad.	Dry	.....	.....	143.7	144.2	141.5	141.6	141.7	140.6	.....	.....	.....	.....
	Plastic	.....	.....	143.0	143.6	141.0	141.1	140.4	140.6	.....	.....	.....	.....
	Excess	.....	.....	143.3	144.2	140.9	141.8	141.9	140.2	.....	.....	.....	.....
	Mean	.....	.....	143.3	144.0	141.1	141.5	141.3	140.5	.....	.....	.....	.....
Empire.	Dry	.....	.....	142.6	144.3	140.5	140.8	139.8	139.7	.....	.....	.....	.....
	Plastic	.....	.....	142.9	144.0	141.5	141.1	141.4	140.4	.....	.....	.....	.....
	Excess	.....	.....	144.6	143.7	141.7	141.7	141.1	139.4	.....	.....	.....	.....
	Mean	.....	.....	143.4	144.0	141.2	141.2	140.8	139.8	.....	.....	.....	.....
Champion.	Dry	142.5	143.1	142.7	140.3	138.9	139.4	.....	.....	.....	.....	.....	.....
	Plastic	142.4	142.8	142.3	141.3	140.6	138.3	.....	.....	.....	.....	.....	.....
	Excess	141.5	142.2	140.5	142.7	139.4	139.0	.....	.....	.....	.....	.....	.....
	Mean	142.1	142.7	141.8	141.4	139.6	138.9	.....	.....	.....	.....	.....	.....

While many of the points brought out by the discussion are of great practical interest, the author wishes to say that many more interesting points could have been touched upon; as, for instance, the

change of volume of mortar material when water is added, the effect of ramming on consolidation of mortar itself, the consolidation or reduction of void space in the aggregate material under ramming, the probable greater reduction of such space with rich mortars which act more thoroughly as lubricants, and many others which could be mentioned. The paper itself was purposely left a bare recital of main facts in regard to the tests, of which the detail is given in the report to the State Engineer and Surveyor, the author's hope being that all the various points of view would be taken up in the discussion. That many of them are left untouched, seems to show, on the whole, a lack of appreciation of the theory of concrete; at the same time the author agrees thoroughly with Mr. Hall that there is no reason why the old nomenclature should obscure, from those who are looking for it, the true theory of concrete mixtures. Nevertheless, the fact remains that apparently either that or some other cause has so far obscured the true theory of concrete mixtures that American engineering literature is, in the last year of the nineteenth century, practically barren of rational information as to the theory of concrete. Mr. Rafter.

Professor Baker's view, that 40% concretes should show an excess of strength over 33% concretes in the same proportion to the excess percentage of mortar, is not supported by the results of these tests. The reason probably is that in 33% concretes the strength is largely dependent upon the close bearing of the broken stone, while in the 40% concretes it is, in a somewhat larger degree, dependent upon the strength of the mortar. Professor Baker, however, is right in his conclusion that, as regards final results, the method of storage is not very important for concrete which has, as in the present case, attained an age of at least a year and a half. Had the blocks been broken at an earlier period, they would probably have shown greater differences than those indicated by Table No. 7.

As to the variation in the strength of the concrete, under the same conditions for the different cements, the author has no explanation to offer, other than that it is possibly due to varying qualities in the cements themselves which have not thus far been closely differentiated by the methods of tests actually in use.

Mr. Le Conte has indicated the true phase of concrete practice at the present time by his remark that "it is a humiliating spectacle to see first-class material, manipulated in an irrational and wasteful manner, developing results which cannot be shown to possess a single redeeming feature." Especially is this true as regards the cleanness of the aggregate material. The broken stone should, in every case, be washed clean, and indeed cleanness should be made an indispensable condition at every stage of the operation. That such conditions are not generally insisted upon, merely illustrates how far many of us are from understanding the true theory of concrete. These remarks

Mr. Rafter. apply to a considerable proportion of the concrete used at the present time in the United States.

The tests recorded by Mr. Von Schon are very interesting, especially as to the general programme which he outlines. His first condition, to operate practically, avoiding the introduction of refinements which cannot be followed in actual construction, is thoroughly in line with the author's views, as is also the second. The author cannot but think, however, that Mr. Von Schon's tests would have been more satisfactory had they been on the line of crushing rather than breaking bars by applying a load at the center, though it is recognized that this is, after all, merely a difference of opinion; the author's contention being, however, that had Mr. Von Schon broken a number of cubes, the results would have been comparable with most of the results thus far obtained, whereas, as the matter stands, it is impossible to compare them.

By way of showing the variations in the quantity of green mortar produced from the same quantity of cement and sand, but with such variation in the quantity of water as is necessary to produce dry, plastic or excess mortars, the following figures are cited:

For 1 to 3 mortars, $\frac{2}{3}$ cu. ft. of cement and 2 cu. ft. of sand produced quantities of mortar in cubic feet, as follows:

	Genesee.	Wayland.	Iron Clad.	Empire.	Champion.	Means.
Dry.....	2.05	2.12	2.19	2.16	2.13	2.13
Plastic...	2.21	2.16	2.23	2.25	2.18	2.21
Excess...	2.25	2.16	2.28	2.29	2.15	2.23

For 1 to 2 mortars, 1 cu. ft. of cement and 2 cu. ft. of sand, produced quantities of mortar in cubic feet, as follows:

	Genesee.	Wayland.	Iron Clad.	Empire.	Champion.	Means.
Dry.....	2.51	2 37	2.57	2.50	2.22	2.43
Plastic...	2.58	2.49	2.62	2.58	2.43	2.54
Excess...	2.75*	2.58*	2.62*	2.61	2.48	2.61

Inasmuch as the cement was weighed, thus insuring the same quantity in each series, the irregularities in these results are chargeable to variations in sand and quantity of water.

The author wishes to thank those who have taken part in the discussion for the many valuable suggestions made. In view of the large number of works, now either in progress or contemplated, in which concrete is either in use or to be used, the author hopes that the general question of the rational relation between the voids in the aggregate material and the amount of mortar, especially in relation to unit costs, and other important questions, may be so thoroughly taken up as to lead to final, definite conclusions. The present paper was intended to be largely suggestive.

* One determination only.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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No. 856.

FRICTION COEFFICIENT FOR RIVETED STEEL PIPE.

An Informal Discussion at the Annual Convention, June 28th, 1899.

Subject for Discussion.

"What is the Proper Friction Coefficient for use in the design of Riveted Steel Pipe?"

By A. McL. Hawks, Jun. Am. Soc. C. E.

A. McL. Hawks, Jun. Am. Soc. C. E. (by letter).—The writer has tested the flow in a 14-in. riveted steel pipe after it had been in service over three years, and again nearly three years later. As these measurements were carefully checked by volumetric determinations after weir flows were measured; as they check very closely with one another, and as they were made upon the main inflow of a system in the ordinary everyday uses to which such a main is applied, they will be a guide to designing similar pipes for such purposes. Mr. Hawkes.

This main is made of soft steel, is 14 ins. clear diameter of the smaller ring, and has double-riveted longitudinal seams and single-riveted circumferential seams. The double riveting is spaced $\frac{45}{64}$ in. between rows, $\frac{54}{64}$ in. to $1\frac{3}{64}$ ins. between heads, and staggered. The circumferential rivets are $\frac{45}{64}$ in. from center to center upon the exterior surface, or about $\frac{1}{2}$ in. from center to center on the interior surface, varying slightly with the thickness of the steel, the gauge of which

Mr. Hawks. runs from No. 9 to No. 12. All rivet heads are circular or "snap heads," and as the diameter of the rivets is $\frac{15}{64}$ to $\frac{21}{64}$ in., the heads form almost a complete ring upon the interior of the pipe, and practically reduce the clear opening about $\frac{1}{4}$ in. An attempt was made to compensate for this by making the interior rings a slight fraction of an inch greater than the size originally planned. As the outer rings were kept uniform, this increased size amounts to about $\frac{1}{8}$ in. in the lighter and $\frac{1}{16}$ in. in the heavier steel.

Seven rings of steel plate (four of the greater and three of the lesser diameter) formed a 24-ft. length of pipe as made up in the shop. One end was reinforced with a 5-in. welded steel ring of 14 ins. clear diameter, the end lengths of the pipe being of the greater diameter and the end longitudinal rivets being practically countersunk to admit this reinforcer. In making up the pipe lengths the longitudinal seams were "in line" in alternate rings and "quartering" in adjacent rings.

All lengths were thoroughly cleaned and dipped in a hot asphaltum bath, being left a sufficient length of time for the metal to attain the temperature of the bath. All lengths were inspected and tested previous to leaving the shop. That this work was thoroughly well done is shown by the fact that some extra lengths and extra rings were left exposed to the weather along the line of the work, and the coating of these was in fairly good condition after more than six years.

In laying the pipe in the trench the longitudinal seams were placed so as to form an angle of 45° with the vertical plane; the reinforcer, which projected half its width from the end of the pipe, was forced into the next length until the exterior rings were in contact. An exterior welded ring $\frac{3}{8}$ in. thick and 5 ins. wide was drawn over the joint and an ordinary lead joint was poured and caulked.

As will be seen from the above, the obstructions to flow are: 1st, the circle of rivet heads of the transverse joints, about $3\frac{1}{2}$ ft. apart; 2d, the lap of the rings; 3d, the reinforcing rings, 24 ft. apart; and 4th, the double line of rivet heads making the longitudinal seams.

The total length of this pipe line is 70 700 ft., leading from an iron screen chamber, near the inlet, to the distributing reservoir. The profile of the pipe line is fairly smooth, there being four principal summits (two approaching the hydraulic grade line) and five principal depressions (two approaching zero of the datum plane); the remainder is chiefly along benches with easy grades. Automatic air valves are provided for all summits and blow-off gates for all depressions. The alignment is very easy, no quick bends occurring in the whole length. In two places where 90° curves are required they are made with six 15° cast-iron pipes. Therefore, no allowance for friction in bends is necessary in the calculations.

This work was completed in the fall of 1892. In April, 1896, the writer tested it, and the following are his notes:

April 2d.—Visited intake and headworks. At 1.30 P. M. measured depth of water in screen chamber, which was 7.98 ft., making elevation of surface of water 429.15 above datum. Mr. Hawks.

April 3d. Weir test.—The weir was a square notch, 15 ins. wide, cut in a sheet of No. 8 steel, set in the overflow chamber of the reservoir. All the conditions surrounding it were favorable to using the Francis formula, with end contractions: Discharge $3.366 \times$ (length of overfall$-\frac{1}{5}\,h$) $\times \sqrt{h^3}$, and this was used in calculating the discharge.

As the total outflow was only 1 cu. ft. per second and the area of the approach channel was 2.8 ft. $\times$ 3.1 ft., or nearly 9 sq. ft., the velocity of approach was disregarded.

Reading on gauge at	8	P. M.	 4	ins.
"	"	" 10	" 4⅝	"
"	"	" 10.15	" ..... 4⅝ +	"

The weir was then closed for 15 minutes to allow the reservoir to fill to a higher level. It was reopened at 10.30 P. M., when the reading on the gauge was 4⅞ ins. It gradually fell until at the close of the test it read 4⅝ + ins., and remained stationary at that figure for 30 minutes.

Elevation of surface water = 396.95 ft. above datum.

April 4th. Volumetric Test.—The supply to the city was cut off at 7.20 P. M.

Gauge reading at	7.30	P. M.	..... 11 ft.	5⅝ ins.		
"	"	7.45	" 11	"	6	"	⅜-in. rise.
"	"	8	" 11	"	6⅜	"	⅜ "
"	"	8.15	" 11	"	6⅞	"	½ "
"	"	8.30	" 11	"	7¼	"	⅜ "
"	"	8.45	" 11	"	7¾	"	½ "
"	"	9	" 11	"	8⅛	"	⅜ "
"	"	9.15	" 11	"	8⅝	"	½ "
"	"	9.30	" 11	"	9⅛	"	½ "
"	"	9.45	" 11	"	9½	"	⅜ "
"	"	10	" 11	"	9⅞	"	⅜ "

The first and last readings were taken most carefully. Intermediates were taken to roughly check the results.

The total rise was 0.35 ft. in 2½ hours.

After this test was completed the water was cut off from the reservoir, and it was left to stand full until morning. As the gauge read precisely the same at the end of 12 hours it was evident that no leakage of moment had occurred during this test.

The flow over the weir was calculated to be 0.96 cu. ft. per second.

Discharge $= 3.366 \times (l - \frac{1}{5}\,h) \times \sqrt{h^3} = 3.366 \times (1.25 - 0.08) \times \sqrt{(3.9)^3} = 3.938 \times 0.244 = 0.96$ cu. ft. per second.

Mr. Hawks. The rate of inflow was 0.99 cu. ft. per second. The average area of the reservoir surface at time of filling = 25 225 sq. ft.

Rise of 0.35 ft. over an area of 25 225 sq. ft. = 8 829 cu. ft.
Allowance for voids in paving..... 65 "
Total..... ..... 8 894 cu. ft.

Total inflow = 8 894 cu. ft. in $2\frac{1}{2}$ hours (9 000 seconds), or at the rate of 0.99 cu. ft. per second.

This rate of discharge agrees nearly with the calculated discharge by Kutter's formula if the coefficient of friction $n = 0.013$ be used.

$$S = \frac{429.15 - 396.95}{70\ 700} = \frac{32.2}{70\ 700} = 0.000455.$$

$$V = C\sqrt{R\ S} = 87\sqrt{0.292 \times 0.000455} = 87 \times 0.0115 = 1.0005 \text{ ft. per second.}$$

Discharge = $1.069 \times 1.0005 = 1.07$ cu. ft. per second.

In February, 1899, the writer was called upon to measure again the flow through the same main, under a slightly greater head. A weir box 22 ins. wide, 20 ins. deep and 9 ft. 3 ins. long, was built at the inflow of the reservoir, having a 90° triangular notch of No. 20 sheet iron at the outflow end. Through this the water flowed 8.9 ins. deep. Using Thomson's formula for the discharge of triangular notches: The discharge = $0.0051 \times \sqrt{h^5} = 0.0051 \times 237 = 1.209$ cu. ft. per second.

A volumetric test was made by allowing the water to flow freely into the reservoir for 4 hours 5 minutes. The average area of the water surface was 13 149 sq. ft., upon which the rise was 1.34 ft.

Rise of 1.34 ft. over an area of 13 149 sq. ft. = 17 620 cu. ft.
Allowance for voids in paving..... 80 "
Total..... 17 700 cu. ft.

Total inflow 17 700 cu. ft. in 4 hours 5 minutes (14 700 seconds), or at the rate of 1.204 cu. ft. per second.

Notes of Volumetric Test.—The supply to city was cut off at 8.30 P.M.

Gauge reading at	8.45 P. M.	2 ft.	$1\frac{1}{16}$ ins.			
"	" 9	"	2	$2\frac{1}{4}$	"	1.15 ins. rise.
"	" 9.30	"	2	$4\frac{1}{4}$	"	2 "
"	" 10	"	2	$6\frac{1}{4}$	"	2 "
"	" 10.30	"	2	$8\frac{1}{8}$	"	$1\frac{7}{8}$ "
"	" 11	"	2	$10\frac{1}{8}$	"	2 "
"	" 11.30	"	3	$0\frac{1}{8}$	"	2 "
"	" 12	M.	3	2	"	$1\frac{7}{8}$ "
"	" 12.50	A. M.	3	$5\frac{9}{16}$	"	3.2 "

At which time the reservoir was cut off for 12 hours. No leakage was observed.

The first and last readings were taken most carefully. Intermediates were taken as rough checks. Mr. Hawks.

This rate of discharge nearly agrees with the calculated discharge by Kutter's formula if the coefficient of friction $n = 0.013$ be used.

$$S = \frac{430.3 - 389.0}{70\ 700} = \frac{41.3}{70\ 700} = 0.000584.$$

$V = C\sqrt{R\,S} = 87\sqrt{292 \times 0.000584} = 87 \times 0.0132 = 1.148$ ft. per second.

Discharge $= 1.069 \times 1.148 = 1.227$ cu. ft. per second.

From these two tests the writer is led to believe that, for similar pipes under like heads, the coefficient of friction, $n = 0.013$, is approximately correct.

He also wishes to draw attention to the fact that there has been practically no decrease in the discharge of the main after it has been in use more than six years. From all the data he can gather, he is of the opinion that, for the first ten years of its life, such a main can be depended upon to give its full calculated discharge.

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No. 857.

STREAM CONTAMINATION AND SEWAGE PURIFICATION.

An Informal Discussion at the Annual Convention, June 28th, 1899.

Subjects for Discussion:

(*a*) "Should Stream Contamination by the Sewage of Cities be absolutely prohibited by law?"

(*b*) "Should the Purification of the Sewage of Cities be Compulsory; and is this feasible for Large Cities?"

(*c*) "Is Filtration the coming solution of the Pure-Water Question for Cities?"

By Messrs. R. E. McMath, W. C. Parmley, Gardner S. Williams, Peter Milne, Kenneth Allen, James Owen, Charles G. Darrach, E. W. Harrison, P. A. Maignen, L. L. Tribus, Palmer C. Ricketts, James S. Haring and James H. Fuertes.

Mr. McMath. R. E. McMath, M. Am. Soc. C. E.—This question has been subdivided into three topics, the first being: "Should Stream Contamination by the Sewage of Cities be absolutely prohibited by law?"

This question may be answered in the affirmative on the ground of the common-law principle: "Each must so use his own as not to injure any other." No individual can discharge filth or other objectionable matter into a ditch or drain which traverses the field of his neighbor. If he attempts to do so the court will afford the injured party a remedy. If a city, town or village defiles a stream with sewage, the

Mr. McMath.

law is an already existing remedy, and has been applied in many cases which could be cited.

The presentation of this topic for discussion by this Society does not imply any doubt as to this recognized law, but, perhaps, is intended to call the attention of engineers to its existence and to the necessity for its extension to questions which, in this country, may arise between the people of adjacent States, if the defiled stream is a boundary, or passes from the territory of one State into that of another. That is to say, is there not a necessity for a general law which will embrace all States, and will settle all questions which arise between States?

Several of the States do exert their powers to prevent contamination of streams by the cities under their jurisdiction, but there is, as yet, no statutory provision to protect against contamination by the sewage of cities in other States. If, by the indifferent or mistaken policy of one State, its cities, great or small, are allowed to defile the streams, must the people of other States suffer for lack of adequate remedy?

The established law referred to has not been enforced in many cases where the transgressor is a city, but the transgression being for a time condoned does not establish a right to transgress for all time. As the country increases in population, the pollution of streams will increase and will quickly pass beyond possible toleration.

As the speaker views the matter, a law more imperative than any statute should control every engineer who has to do with the sewerage plans of cities; and he should foresee and provide for the disposal of sewage eventually, if not immediately, in another manner than by discharge in crude form into streams. Anything short of this is not worthy to be called a plan for a sewerage system.

The City of Chicago has for years been preparing a channel for the discharge of its sewage through its back door, so that this sewage will pass the front doors of other cities, and some of these other cities are not well pleased by the prospect, especially since the demonstration that certain diseases are water-working and that the pathogenic bacteria are tenacious of life. The topic under discussion would be an important one if Chicago did not exist, but the Chicago case gives much emphasis to the topic at the present time, for it is a scheme to do permanently, and on a large scale, what everyone must admit should not be done at all—a scheme which is to-day without an avowed designer and almost without a defender.

There was a time when many believed that the water of a river purified itself. Others firmly believed that dilution to a point beyond the reach of refined chemistry must render such water innocuous. The Chicago scheme had its origin when these beliefs were held. This scheme, involving the expenditure of more than $30 000 000, has progressed, notwithstanding the passing of the faith upon which it was founded, and now the only plea which is advanced for its consummation

Mr. McMath. is the fact that $28 000 000 and more have been expended on it and it is now too late for objectors to state their objections.

This brings up the consideration of the second topic: "Should the Purification of the Sewage of Cities be Compulsory; and is this feasible for Large Cities ?"

The answer to the first part of this proposition follows that given for the preceding topic. If streams are not to be contaminated, the sewage must be purified; for water is the only known vehicle which can carry the wastes of human life and industry away from our homes and shops. That such can be purified, so that the effluent may be entirely unobjectionable, has been demonstrated beyond occasion for further discussion. How to do it, in each case that may arise, is the work of engineers, to plan, execute and operate.

There is no casuistry in the assertion that in this matter the end justifies the means, even when the difficulties surrounding the end require large and many means.

As to its feasibility for large cities there are too many old-world examples of successful solution for American engineers to hesitate as to the answer. It can be done, and we are the men who can do it.

The third topic comes in to round out the discussion: "Is Filtration the coming solution of the Pure-Water Question for Cities ?"

Asked in connection with the preceding topic, the question may be taken as a suggestion that the sufferer from sewage contamination of water supply must protect himself, and thus relieve his up-stream neighbor of his obligation.

To meet this suggestion it is sufficient to mention the fact that the sufferers do not all live in cities, and are not able to protect themselves. There are evils from such contamination not measurable by the results of a bacteriological examination for specific microbes. There is also a contamination of water by the drainage of lands outside of cities which sooner or later will justify and even compel resort to filtration of the water supply of cities.

The doctors may say that they can cure a large percentage of cases of diphtheria, for instance, but the wise mother does not expose her children to infection on the chance that all cases in her family will be cured.

Therefore, filtration may promise a remedy for defiled waters, but that does not imply that the filtered water would not be better and safer had it not been contaminated.

Mr. Parmley. W. C. Parmley, M. Am. Soc. C. E.—The question of stream pollution is one that concerns almost every city. If no pollution of the water courses is to be permitted, then all the sewage must be purified, and the question resolves itself into one of expense. Small towns, or those of larger size, which are favorably situated, may adopt some method of ultimate treatment at comparatively small cost, but in the

case of many, if not indeed most, of the larger cities, the expense involved by such treatment would be a very great burden, if not altogether out of the question. Insistence upon absolute freedom from pollution would make the interception of sewage and the overflow of storm water an impossibility. Mr. Parmley.

The whole matter is one in which the ideal is set over and against the practicable—the desirable but extremely difficult to attain, against the less desirable but practically possible—but it is no more important that all the water we drink should be free from impurities than that the air we breathe and the food we eat should be pure and wholesome. To insist, however, upon a high degree of purity in a stream from which no drinking water is drawn, or at most only stock water, is to require heavy payment for very small benefits, and many streams have long since ceased to furnish even the latter supply.

The only other cases necessary to consider are those in which the odors arising from a polluted stream become unpleasant or harmful, or where the sight of turbid or colored waters or the refuse left by them along their banks is objectionable for æsthetic or sentimental reasons. The sanitary welfare of the people within the area affected is the ultimate and only condition to be insisted upon.

Even in the earlier development of our cities the natural drainage courses soon become so polluted by local discharge that no drinking supplies are likely to be taken from them, and it remains to be determined whether the pollution which has already occurred shall be removed or not. As most of this pollution is from factory wastes or from the washings of stables and privy vaults, it is a source of impurities nearly impossible to avoid. While the drainage area remains suburban, the cost of adequate sewerage would be too expensive to be borne by a sparse population or an isolated manufacturing company, and when the density of population becomes sufficient to make thorough sewerage possible, the soil and water-courses have already become so polluted that many years would be required to thoroughly drain them. Even then, as those who have had experience will testify, it is impossible to prevent entirely the clandestine discharge of some sewage into the streams. It follows, then, that the smaller courses which are not entirely taken into the sewerage system will have a considerable degree of pollution. Just the percentage which may be allowed is impossible to state, but it is dependent on the character of the polluting matter, the climatic and many other conditions. It is safe to say, however, that no stream should be permitted to become so befouled as to give off offensive effluvia and gases, thus showing that it contains a large amount of putrefying matter. Either, these streams should be diverted into the sewers, or the whole stream taken up in a large trunk sewer.

The question of permitting storm water from sewers to overflow into

Mr. Parmley. water-courses is important in many cities. If such overflows are not allowed, the cost of the sewers thus required, over that where the overflow system is permitted, becomes an item of no mean proportions. The engineering department of the City of Cleveland has at the present time a contention with the Board of Park Commissioners regarding the overflow of storm water from a sewer into a stream; and if it is finally settled that the sewer cannot be permitted to overflow, the cost of the sewer will be about $250 000 more than if it were constructed with storm-water overflows. In another instance the local court has issued a restraining order to prevent the discharge of excess storm-water into a stream which is already highly polluted. The final settlement of the case is of great importance, as upon this decision will depend other important projects in the sewerage of that city. The court apparently ignored the important facts brought out in the trial, viz., that this overflow would not operate except in case of excessively heavy storms, when the water would be discharged for perhaps 15 or 20 minutes, that these overflows would probably occur only a few times during the season, that the actual stream in time of flood would be nearly as impure as the water overflowed from the sewer, and that even if it were not, the time of maximum flood in the stream would be after the sewer had ceased to overflow and the stream would continue to flow with a large volume for many hours afterward, thus washing away the last traces of any fancied pollution.

If sewers are not allowed to discharge or overflow into natural water-courses, in many instances the double system of sewerage will become an absolute necessity. If the sanitary sewage be conveyed to a proper outlet through separate sewers and all the storm-water is to flow to the natural drainage channels, either through storm sewers or directly over the ground, the water discharged therefrom will usually be more foul than that which reaches them from the storm overflow structures of the combined sewers. As all engineers have probably noticed, the first washings of the street surfaces are extremely foul, and, by the system of overflows, this foul water is carried to the sewer outlet while a portion of the cleaner washings which occur later in the storm, mixed with the normal volume of sewage, will escape from the overflows into the stream.

If, however, the contaminating effect of street water be neglected and only the effect of sewage from the sanitary flow be considered, and if the entire volume of dry-weather flow in the sewer be defined as sewage, in the first case above cited it was shown that the maximum degree of pollution, that is, when the sewer is just at the point of overflowing, is 1 of sewage to about 7 of street water, and when the structure is discharging its maximum volume of overflow the proportion is 1 to sewage to 30 of street water. Now, consider the degree of pollution of the stream into which this discharge is made, and the

proportion of pollution is much less in proportion to the volume of water already flowing therein. Mr. Parmley.

It is a matter of the simplest observation to show that in many cases the water escaping from overflows in time of storm is not more foul than that in the swollen waters of the flowing stream. It being thus impossible to prevent the pollution of small courses flowing through thickly populated areas, it seems unwise to attempt to preserve in them a greater degree of purity than that required to prevent their contaminating the surrounding air, and when they are so short, or their velocity of flow is so rapid, that an outlet is reached at a point beyond which it is unnecessary to consider their impurities, and before those impurities have had an opportunity to decompose, the further expenditure of money, in order to preserve a high degree of purity, would be a needless waste.

The matter then suggests the following conditions, with their appropriate treatment:

1st.—Streams or bodies of water from which potable supplies are drawn should be preserved in the purity required by the most advanced sanitary science, and if such purity can be obtained in no other way, the supply should be properly purified before being used.

2d.—Streams or ponds which have sluggish motion with consequent tendency to stagnation should be kept as free as possible from sewage pollution in order that their emanations may be less dangerous.

3d.—Streams in which the flow is so rapid, or in which the courses are so short, that stagnation and decomposition are impossible within the limits of time necessary to consider, may be used advantageously for the overflow of excess storm water, or as the direct outlet for a limited number of sewers.

Gardner S. Williams, Assoc. M. Am. Soc. C. E.—It would be interesting to determine, if possible, where to draw the line as to the limit of the stream which is not used as a source of potable water. Nearly all streams flow into others or into lakes, and at some point drinking water is being taken from them. Mr. Williams.

An interesting case came under the speaker's observation, while he was connected with the water supply of Detroit, Mich., which involved the investigation of a serious typhoid epidemic in that city in 1892.

Port Huron is situated at the head of the St. Clair River and at the foot of Lake Huron. The distance by water from Port Huron to Detroit is about 60 miles, about 18 miles of which is through Lake St. Clair, where the current is very slow, and which acts as a monstrous settling basin, giving ideal conditions for sedimentation and the consequent purification of the water. The sewage of Port Huron is discharged into Black River, a very sluggish stream, which flows through the center of the city and receives sewage from both sides. The current in Black River is practically nothing, except in time of

Mr. Williams. freshet. No water for potable purposes is taken from this stream, but the Detroit and St. Clair Rivers and Lake St. Clair are sources of supply.

In 1890, Congress decided to improve the navigation of Black River, and work thereon began in 1891. Dredging was commenced at the bar at the mouth of the river, and was approaching that part of the river where the principal sewers discharge when cold weather stopped the work. On April 16th, 1892, work was recommenced at a point about 300 ft. below the first bridge, and at that bridge two large sewers discharge into the river. Below that bridge it is reported that a depth of 12 ft. was dredged through an accumulation of sewage deposits which had been gathering there for years. The inspector's records of the dredging show that there were excavations in such material 8 ft. in depth. This material was put in dump scows, taken out into the St. Clair River and dumped at a point over the Grand Trunk Railroad tunnel, or approximately 60 miles above the intakes of the Detroit Water-Works, which are just below Lake St. Clair, and virtually at the head of the Detroit River.

The record of typhoid fever in Detroit at that time is interesting. There had been some deaths from typhoid early in the year, but between May 11th and May 28th, 1892, no deaths were reported. On May 28th there was one death. On June 5th there were four, and during the ensuing 25 days there were thirty-seven deaths from typhoid fever, the population of the city being then about 230 000. The death rate from typhoid in that year in Detroit rose as high as the typhoid death rate of any large American city, except Chicago, Pittsburg, Jersey City and St. Louis, in the past ten years. There were three years in which the death rate in Pittsburg and two in which that of Chicago had been higher.

It was somewhat difficult to trace the cause of this epidemic, and when the speaker began his investigations he was not aware of the operations of the Government in Black River in 1891 and 1892. He therefore did not look there for the pollution, but began examining the river and studying rainfall, the effect of thaws, the effect of ice in the river and the character of the water. None of the ordinary methods of accounting for the presence of typhoid was applicable in this case.

The precipitation during April had been heavy, but it had also been heavy at many other times when there was no typhoid. During that summer all the river cities had typhoid, and in all cases the disease appeared at about the same time. At Port Huron, while there had been a tremendous excess of typhoid late in the fall of 1892, there were only one or two cases in the spring of that year; this pointed to a cause at or below Port Huron.

Finally, the speaker's attention was called to the Government

dredging operations and to the place where the material was dumped. Mr. Williams.
The flow of the Detroit River is given as 225 000 cu. ft. per second. From all available information as to the velocity of the water, it was found that it would reach Detroit in not less than 6 and would probably require not more than 10 days. From a careful study of the statistics of typhoid fever, as published by various Boards of Health, the conclusion was reached that the average period of incubation was 14 days. An examination of the statistics as to this disease showed that death, when it ensues, ordinarily occurs between the twenty-first and twenty-eighth days, which would give 25 days as an average. From April 16th, the day on which the first scow load of polluting material from the sewers of Port Huron was dumped into the St. Clair River, until June 5th, when there were four deaths from typhoid in Detroit, is 50 days. If 10 days are allowed for the water to flow, 14 days for the disease to incubate and 25 days to kill, the total is 49 days.

The investigation was carried on throughout the summer, and the curves of secondary and tertiary infection were studied, and finally, about seven weeks after cold weather set in and dredging operations were suspended, the typhoid ceased. Next year the typhoid appeared again after the dredging had commenced, and when it stopped the disease disappeared. These conditions have followed one another since that time, and investigations have shown that they prevailed in previous years. Going as far back as 1886, it was found that typhoid appeared during those years when dredging operations disturbed the channel of the St. Clair River or Lake St. Clair above the intake of the Detroit Water-Works, even as far away as Lake Huron; and the relationship could be traced accurately. In one or two years, when there has been no disturbance of this channel, Detroit has had a very low typhoid death rate.

Another interesting case occurred in 1897, which was a year of very low typhoid in Detroit, the rate being only 1.1 per 10 000 living. During that year there was only one group of cases which could be considered as epidemic, and that came in August. In two weeks there were seven or eight deaths. In looking for the cause it was found that the Government was dredging in the Clinton River. The work had been commenced some time before, and the dredge had been working gradually up the river, the material being dumped in Lake St. Clair, in what was supposed to be stagnant water, at a point about 16 miles from the water-works intake. At a certain date the dredge arrived at a bar in the river just below Mount Clemens, a town which, though not having a complete sewerage system, has a few sewers discharging into the Clinton River. A computation, similar to that previously described, showed that the group of cases in August was due to infection stirred up by these dredging operations.

Mr. Williams. These instances are given for the benefit of those who speak of streams which are not used as sources of water supply. There is no stream which is not a source of water supply for some person or community, and there is no danger of going too far in the restriction of the pollution of our streams.

Mr. Milne. PETER MILNE, Assoc. Am. Soc. C. E.—The American Water-Works Association has had the subject of stream pollution under consideration for the past eight years. Judging from correspondence with State Boards of Health and eminent medical authority, the speaker is led to believe that interstate legislation will not accomplish the object sought; and that it will be necessary to appeal to the Congress of the United States to seek the enactment of national laws to prohibit the pollution of water supplies. Such law may of necessity guard riparian rights and also protect the health of the people using the water of streams liable to be contaminated by sewage of every character.

A special standing committee of the American Water-Works Association has nearly ready for publication a voluminous report upon this subject.

An expression of opinion upon this important question by this Society will carry weight and influence and be accorded the consideration due.

An example of the manner in which a community may recede from its position taken regarding sanitation of water supply, is afforded by the experience of the City of Brooklyn, N. Y. More than four years ago, two streams, which formed a part of the daily supply, were condemned by the Board of Health of the City as unfit for use, on account of pollution due to the sewage of the population dwelling upon the water sheds, which pollution had increased yearly in proportion to the increase of population. These stream supplies were abandoned. In the present emergency and extremity of the city, viz., a long drought, it is suggested to again use the water of these streams, provided it is filtered.

The question of filtration being one of degree, it is apparent that its range of effectiveness in application is contingent upon the measure of pollution to be met and treated. If pollution is permitted to increase in intensity as well as quantity, it is reasonable to assume that the cost of filtration would be prohibitory and compel the consideration of new sources of water supply remote from the influence of pollution.

If the Federal Government enacts laws which in effect restrict pollution and contamination of the waters which are used by the people, it must invoke the skill of the engineer to aid it, on the one hand, and wise legislation, on the other hand, in order that equitable relations may be established and that the rights of all interests may be safeguarded in the effort to prevent the defilement of water supplies.

KENNETH ALLEN, M. Am. Soc. C. E.—In studying questions relating to stream pollution it is natural to turn to England, where, in official reports on the subject, there is a storehouse of information which is not to be found elsewhere; although of recent years the investigations undertaken by several States—notably Massachusetts—and by several cities, have made available much information of the greatest value. Increasing density of population, the concentration of large industries, and a general awakening of the masses to the importance of their sanitary environment, all serve to direct attention to the matter of stream contamination. Mr. Allen.

Up to the present time, the most important legislation on the subject has been the Rivers Pollution Act of 1876, resulting from a report of the Second Rivers Pollution Commission on the Rivers Mersey and Ribble, in which the following requirements, especially regarding organic pollution, which had been recommended by Dr. Frankland, were adopted:

"We suggest that the following liquids be deemed polluting and inadmissible into any stream:

"(*a*) Any liquid containing in suspension more than 3 parts by weight of dry mineral matter, or 1 part by weight of dry organic matter, in 100 000 parts by weight of the liquid.

"(*b*) Any liquid containing, in solution, more than 2 parts by weight of organic carbon, or 0.3 part by weight of organic nitrogen in 100 000 parts by weight.

* * * * * * *

"(*f*) Any liquid which, after acidification with sulphuric acid, contains, in 100 000 parts by weight, more than 1 part by weight of free chlorine.

"(*g*) Any liquid which contains, in 100 000 parts by weight, more than 1 part by weight of sulphur, in the condition either of sulphuretted hydrogen, or of a soluble sulphuret."

Although adequate provision for the enforcement of this Act appears not to have been made, its requirements were probably as fair and sound as was possible with the scientific knowledge of that time.

In the same year, 1876, the Massachusetts State Board of Health presented, in its annual report, a number of recommendations regarding the discharge of crude sewage into streams; among them:

"That no city or town shall be allowed to discharge sewage into any water-course or pond without first purifying it according to the best process at present known, and which consists in irrigation; provided that this regulation do not apply to the discharge from sewers already built, unless water supplies be thereby polluted; and provided also that such intended discharge can be shown to be at such point or points that no nuisance will arise from it."

But they are careful to add, that:

"The Board feels that, in the present state of our knowledge, sweeping laws for the general and immediate purification of all our streams would be hardly justifiable, and that they are not called for by the present condition of our rivers."

Mr. Allen. In the 23 years which have elapsed since these views were officially expressed, there has been opened up a vast field of research in sanitary matters, through the science of bacteriology, and, as our knowledge of the life history of these minute organisms is as yet quite imperfect, we are not likely to arrive at the ideal treatment of the matter for some years to come.

The standard of purity for sewage effluents, as established by the Second Rivers Pollution Commission, is faulty in the following particulars:

In the first place, the nuisance caused by a foul effluent depends quite as much upon the character of the stream into which it flows as on the effluent itself. A foul effluent discharged into a deep, swift-running stream of well-oxygenated water may cause no offence, while an effluent of half the size and twice the purity of the first, meandering over mud flats to a sluggish brook or mill pond may prove highly objectionable.

It is, therefore, seen that, if a standard of purity for the effluent is to be fixed, it should be in some way correlated to the character of the stream itself, and this under its most unfavorable conditions, as at extreme low water, when covered with ice, or when, for some other reason, it is particularly foul or devoid of absorbed air.

The two following examples serve to illustrate the inconsistency in the standard alluded to.

At the Leeds Congress of the Sanitary Institute of Great Britain, in 1897, Professor A. Bostock Hill, wishing to accentuate the difficulty of attempting to fix a standard for sewage effluents, quoted a case:

"Where a tank effluent, without any preliminary treatment, gave better results than the Rivers Pollution Commission standard, and yet subsequently so seriously polluted a stream that an injunction was applied for and obtained."

On the other hand, Alderman J. F. Spence (Chairman, Tyne Port Sanitary Authority), said:

"Between Tynemouth and Newcastle-on-Tyne, a distance of ten miles by the river, there is as large a number of works discharging foul and poisonous effluents as probably on any other stream in the kingdom. But these are so largely diluted by the enormous influx of sea water twice in the 24 hours, the difference in time between high water at the bar and Newcastle Quay being only 17 minutes, that the late Dr. Frank Buckland, when inspector of salmon fisheries, quoted the Tyne as one of the best salmon rivers in the kingdom, the pollutions being so diluted that the fish passed there unscathed to their spawning grounds in the higher reaches of the river. It may therefore be said the pollutions cause no nuisance, as the tide clears them all away."

Again, the Rivers Pollution Commission standard fails to appreciate the potential capacity for further purification in the effluent itself. This important point has been brought into prominence by the difficulty in securing permission, under existing laws, to discharge

the effluents from certain bacterial treatment plants into neighboring streams, although it is generally admitted that no nuisance would result thereby. The outcome of this is that a Royal Commission has been appointed, and is now giving this matter careful study, which will no doubt end in placing before the world most valuable and complete information on this subject. Mr. Allen.

The final purification of sewage, which is due to the mineralization of its organic constituents, is effected by the action of minute organisms, either vegetable or animal, but generally by bacteria, and this is the case with crude sewage, the effluents from chemical precipitation works or from sewage farms, as well as from the septic tank or other of the methods known more specifically as bacteriolytic. To interrupt this action by the use of antiseptics simply postpones the inevitable and desired result, allowing organic matter to accumulate. Such interruption is, therefore, fundamentally wrong.

The process of mineralization may be roughly considered as taking place in two stages.

In the first stage, the solids are broken up and thrown into solution by liquefying bacteria, while the rearrangement of the molecules of all the organic matter renders it more susceptible to further change. This action is called bacteriolysis, and it takes place in a comparatively short space of time, the characteristic product being carbon-dioxide. Although usually occurring under anærobic conditions—when it is often accompanied by the evolution of sulphuretted hydrogen, marsh gas and other offensive or dangerous compounds, and sometimes, very likely, by the formation of those virulent poisons known as ptomaines —it is pointed out by Dr. Samuel Rideal as quite probable that it may result from the action of such ærobic liquefying bacteria as *B. Prodigiosus* and *B. Liquefaciens* under suitable conditions.

In the second stage, which is marked by oxidation of the nitrogen, and known as nitrification, the bacteria essential to the process must be supplied with oxygen, and it is characteristically an ærobic process.

It is true that in the formation first of nitrites and then of nitrates the oxygen required for the former may, in the absence of free oxygen, be wrenched from the nitrates or from other combinations of oxygen, leaving the final conversion into nitrates to be effected later.

Dr. Walter E. Adeney, of Dublin, has taken advantage of this action in devising the method of sewage and sludge purification known as the "Oxygen System," in which nitrification is effected by the application of nitrate of soda or permanganate of potash, substances which are capable of furnishing the oxygen required.

Nitrification is a comparatively slow process, and, while accomplished in an ideal manner by intermittent filtration, yet by reason of its slowness it may often be effectively carried on in streams by means of the oxygen dissolved therein. When, however, this element is not

Mr. Allen. absorbed from the atmosphere as rapidly as it is taken up by the organic matter undergoing purification, anærobic conditions prevail, which we call putrefaction, with their attendant evils.

It is clear, then, that the ability of the stream to oxidize the impurities in an effluent is a most important consideration, which has been quite ignored in the standard set by the Second Rivers Pollution Commission.

The determination of the organic nitrogen and carbon, as proposed by Frankland, has been found difficult in practice, and has been largely superseded by the less precise but more practicable determination of (or test by) albuminoid ammonia, as proposed by Wanklyn, or of the oxygen consumed, as proposed by Forschammer, or by both of these together.

Messrs. Pearmain and Moor have recently proposed the following standard, as adapted to present needs:

	Parts per 100 000.
Total suspended matter (not to exceed)	2.0
Albuminoid ammonia "	0.2
Oxygen absorbed "	0.714
Total hardness "	28.57
Nitrogen as nitrates (not to be less than)	0.714

They insist on the last requirement for the reason that, besides indicating the completion of the process, the nitrates also indicate a store of oxygen which may be drawn upon to oxidize the remaining impurities, insuring thereby a guarantee against putrefaction.

Dr. Rideal, to accomplish the same end, has suggested, as a safer test than any previously proposed, "the ratio of the oxidized nitrogen in the form of nitrate and nitrite to the 'oxygen consumed' figure," as the former represents the oxygen available and the latter the oxygen required to carry out the process of purification. He states as his belief that when the available oxygen is in excess,—

"Then under no subsequent conditions would a reversion of the sewage take place so as to cause a nuisance, and such a method of valuing the suitability of an effluent was independent of the size of the stream or the dissolved oxygen it contained. When, and in the majority of cases this was the case, the stream was many times the volume of the sewage and contained dissolved free oxygen, then this free oxygen was utilized instead of the oxygen of the oxidized nitrogen. Under these conditions it was not necessary for the oxidized nitrogen as compared to the 'oxygen consumed' figure to be so large."

If the character of a proposed effluent, as well as its probable dilution in running water, could be predicted with certainty, its liability to cause offense could be pretty well foretold. A rough but convenient limit was determined in this way by F. P. Stearns, M. Am. Soc. C. E., when Chief Engineer of the Massachusetts State Board of Health, substantially agreeing with a similar limit fixed by Rudolph

Hering, M. Am. Soc. C. E., with reference to Chicago's sewage discharge, in which he designates a stream flow of not less than 2½ cu. ft. per thousand population as sure to cause offence, and a flow of over 7 cu. ft. per thousand population as not likely to cause offence. Mr. Allen.

Such a rule will, of course, apply only when manufacturing wastes are negligible in quantity or character; but where questions involving the dilution of crude sewage are concerned, this method of predicating offensive conditions is likely to prove as convenient and satisfactory as any, considering the number of indeterminate factors involved.

Whether more definite tests, to cover various conditions, are practicable remains to be seen, but the more recent ones appear to be steps in the right direction. Possibly the future may discover some simple biological tests by noting the effect of polluted streams upon various kinds of fish or other organic life contained in them. In fact, in the elaborate series of experiments on sewage disposal which has been carried on for several years past at Leeds, one test of certain effluents was their capacity to support the life of carp, the effect on confervoid growths being also noted. It is known that different kinds of fish, such as salmon, bass, catfish and menhaden, require waters of different degrees of purity, and their survival would, to a certain extent, be a measure of the contamination of the water in which they were placed.

A great economic advantage in bacterial treatment is the elimination of the vexed question of sludge disposal. It is true that sand and other grit still have to be removed, and that woody fibre requires time to become peptonized without clogging the filter, but the troublesome and offensive organic solids may be effectually disposed of, and this is a great step in advance.

The relative efficiency of such treatment, as compared with mechanical filtration and chemical precipitation, is well illustrated by the following results obtained at Leeds:

	Oxygen Absorbed in 4 Hours.		Albuminoid Ammonia.	
	Parts per 100 000.	Percentage purification.	Parts per 100 000.	Percentage purification.
Crude sewage	12.14	....	1.21	....
Filtration through paper	5.53	54	0.67	45
Lime precipitation	6.57	46	0.64	47
Bacterial action (Dibdin's filters)	0.91	92	0.125	90

Crude sewage contains several million bacteria per cubic centimeter —London sewage some 3½ million. After complete nitrification, these are reduced to an insignificant number; but before that time, although an effluent may contain sufficient oxygen to complete the process of purification, it may contain a very large number of these organisms.

Mr. Allen. Now, while there is some evidence to indicate that pathogenic varieties will have been eliminated before arriving at this stage, yet with the present imperfect knowledge, and with the possibility of their subsequent introduction into an effluent yet containing nutrient material, it would probably be wise to fix a low limit to the bacteria per cubic centimeter permissible in a stream used for a water supply which has received the effluent from biological filters. In fact, it is a question whether the water of such a stream should not be filtered before distribution to a city, as there is always a chance that at times, through accident or excessive flow, the purification of the sewage may not be up to the established standard.

When the stream is not used for drinking purposes, the presence of a large number of bacteria in the effluent is not necessarily an unfavorable indication.

It is believed that, for the present, legislation regarding stream contamination and sewage purification should be largely tentative, both because experience in these matters is as yet limited, and because it is rapidly increasing.

In dealing with a subject so complex and, at the same time, often involving the public health and the outlay of large sums of money—in the establishment of purification works on the one hand and the depreciation of riparian property on the other—each case of pollution should be considered by itself, by an authorized official, who should be a man of much tact and ability, as well as one having special knowledge and experience in his line. He should be guided by certain general laws regarding stream pollution, but should be given considerable discretionary power, and, to avoid troublesome changes of policy, it would probably be well to have him serve a long term of years. On the other hand, he should not be enabled to use his authority in too summary or arbitrary a way, to the disturbance of vested interests, when not absolutely necessary for the public good.

In order to make his supervision effective, the area under his control should be defined by the natural water-shed lines; that is, he should have authority over the entire drainage area of one or more streams, and be responsible for their condition.

In order to effectively control stream contamination, it is suggested that all streams may be placed in one of three general classes, viz.:

I. Streams far from populous centers and not likely to be used for manufacturing or domestic purposes.

II. Streams likely to be used for other than domestic purposes.

III. Streams likely to be used for domestic purposes.

Such a classification would not appear to be difficult or intricate, and laws regarding permissible pollution would apply, in general, to all streams of the class in question. Class I would require little or no legislation, while certain standards of purity for sewage effluents,

etc., would apply to Class II and others to Class III. Moreover, in the case of large rivers, like the Mississippi or Hudson, it would be quite possible to include the headwaters and upper reaches in Class III and leave the lower reaches in Class II. On the other hand, if certain populous centers toward the mouth of the stream should befoul its waters to the detriment of the fishing or other interests near by, it might be necessary to create a special district to which the restrictions for Class III would apply. Mr. Allen.

That the purification of the sewage of large cities by land disposal is feasible, is seen by such examples as Berlin and Paris.

Suitable land, however, is not always available, when chemical treatment, followed by filtration through the natural soil, as at Birmingham and Manchester, or through prepared filter beds, as at Glasgow and Sheffield, may be resorted to. In the cases of London and Salford, the sewage is merely clarified by chemical precipitation, and the sludge is carried out to sea by steamers and dumped; and in such cases the effluent cannot be said to be truly purified. The previous examples, however, suffice to show that there is no inherent difficulty in purifying the sewage of a large city, even if irrigation cannot be availed of. It is merely a question of selecting the best and cheapest.

The modern bacteriolytic processes avoid the acquirement of extended areas for irrigation on the one hand and the question of sludge disposal on the other, and from them the ideal solution of the sewage disposal question of the future may be hoped to be realized. Experience has already been sufficient to obtain for these methods very high endorsement.

For instance, in testimony before a Local Government Board Inquiry at Exeter, regarding the septic tank treatment, Dr. Rideal gave as his opinion, that

"Wherever a sewage farm would work, this system would work, and anything which would prevent this system working would prevent a sewage farm working."

When asked whether it could be applied to very much larger towns, he replied "much larger." Dr. Dupré, also, declared that "no system of purification hitherto known had such a complete effect upon sewage."

It must be admitted, however, that up to the present time no great city uses any of these methods. The largest plant is at Sutton, England, where some 145 000 U. S. galls. per day are treated by the Dibdin system of double filtration.

The Sewerage Commission of the City of Baltimore, which for some time has had this very matter under consideration, while admitting the admirable results attained at Sutton, Exeter, and elsewhere, and after consultation with Mr. James Mansergh, regarding this particular point, has refrained from recommending any one of these sys-

Mr. Allen. tems for so large a city, involving, as it must, much that would be new when applied on the scale proposed. But this decision is qualified by the statement that

"The Commission fully recognizes that one or more of them may prove available for the treatment of the sewage of some of our outlying districts, and may, if future experience sustains the claims and anticipations of their advocates, yet become adequate to our service on a larger scale."

To sum up what has been said, it is believed:

(*a*) That stream contamination should be prohibited, and the purification of the sewage of cities be compulsory only:

(1) When it is offensively evident to the senses of smell or sight.

(2) When manufacturing, fishing, or other productive interests are likely to be seriously affected thereby.

(3) When the health of neighboring communities is jeopardized, either through the authorized use of the stream for domestic purposes, the probable contamination of areas devoted to the culture of shell-fish, or the probable formation of deposits upon shoals or shores

(*b*) That the purification of the sewage of large cities is always feasible. It is always costly, but it is probable that the introduction of bacteriolytic methods of treatment will, in general, when more fully developed, tend to economy.

(*c*) That to make legislation effective:

(1) All streams should be classified according to their permissible contamination.

(2) A responsible officer should supervise and control all sources of pollution throughout the drainage area of every stream in which pollution is restricted by law.

(3) Where possible to formulate consistent regulations regarding the limits of contamination, this should be done; but that with regard to organic contamination, in the present imperfect but advancing state of our knowledge on this subject, the matter may best be left largely to the discretion of the official in charge of the particular drainage area in question.

Mr. Owen. JAMES OWEN, M. Am. Soc. C. E.—It has not been regarded as within the province of this Society to discuss legal questions, but, recognizing legal enactments as the crystallization of public opinion on the subject legislated upon, the legal aspect of stream pollution is worthy of consideration. The radical, unqualified proposition that all pollution of streams should be summarily stopped seems to the speaker unwise. The question is a very live one in New Jersey, and the conditions in the northern part of the State furnish an object lesson.

Mr. Owen.

The Passaic River was formerly known as "the beautiful Passaic;" now it is called "the dirty Passaic." Along its banks is a population of about 400 000 persons, who, by the construction of sewers, have completed, within the past twenty years, a system of discharge which renders the river unfit for potable uses, or for any purposes for which water can be used. Until three years ago, Jersey City took its water from the lower Passaic. It was bad water, much contaminated and unfit to drink, and Jersey City vainly tried, for years, to obtain from the Courts means to restrain the towns above its intake from discharging their sewage into the stream.

Newark obtained a new water supply about ten years ago, and Jersey City got a new supply three years ago. Curiously enough, as soon as Jersey City stopped taking its drinking water from the river, there arose a popular demand that the pollution of the river should cease. Paterson, with 100 000 population, is discharging a large amount of sewage into the river, and Newark and the towns between Newark and Paterson wish defilement from this source prohibited. Newark also discharges an enormous amount of sewage into the river, and this drifts up with the tide for four or five miles above the city.

On the Upper Passaic is the small town of Summit, which is discharging into the river, not crude, but filtered, sewage. The City of Paterson has brought suit against Summit to prevent its pollution of the stream. As a kind of cross-fire comes the proposition of the City of Paterson to enjoin the East Jersey Water Company from taking water from the river, on the plea that there will not be left in the river sufficient water to take the city's sewage down. From this it will be seen that the trend of public opinion in that locality is indefinite.

It cannot be too strongly emphasized that there should be no contamination of drinking waters or the sources of such waters; but the matter of the discharge of sewers into rivers used merely for navigation, or for drainage, should be considered as a question of dilution; and if, within the next few years, a standard of permissible dilution, which would be free from effects deleterious to the community, can be established, the prime object of this inquiry will have been attained.

Mr. Darrach.

Charles G. Darrach, M. Am. Soc. C. E.—The study of this problem for twenty-five years leads the speaker to agree with the conclusions so admirably expressed by Mr. McMath. The engineers of America are competent to solve this problem, and the speaker anticipates not only its solution but such a development in the future as will modify the water-supply question to the extent that if necessary a city will not be obliged to go beyond the purification of its own sewage to supply itself with water.

Mr. Harrison.

E. W. Harrison, M. Am. Soc. C. E.—The subject of the right of cities on tidal streams to discharge their sewage therein has not been taken up in this discussion, but some recent events indicate a disposi-

Mr. Harrison. tion to impose unnecessary and harmful restrictions on such communities. When the City of Newark was founded, about 200 years ago, the site was chosen because it was on a tidal stream. It offered, by reason of its location, advantages for settlement, for manufactures and for commerce. This agitation for the purification of the Passaic River, of which Mr. Owen spoke, resulted in swinging the pendulum far from the stand taken by the Courts ten years ago, for in March last the City of Newark was enjoined from draining about 130 acres of land into the Passaic at a point where the tide ebbed and flowed. The water at this point was salt, though not as salt as the sea, but both medical testimony and a bacteriological examination were submitted to show that the growth of bacteria was inhibited by reason of its saltness. The river was thus acting only as a carrier, conveying sewage to the sea without affording opportunity for the further growth of bacteria.

That agitation and decision are now bearing strange fruit. Other towns draining into estuaries of New York Bay are seeking places of deposit for their sewage, and they can keep their water supplies pure only by turning their sewage into the tidal streams. This decision has helped to create the opinion in northern New Jersey that no sewage should be emptied into any stream, tidal or otherwise, not even into the Kill von Kull, a tidal strait between Staten Island and New Jersey, passing an enormous quantity of water with a swift current.

The recognition of this argument as a governing principle would operate to deprive cities located on estuaries and tidal streams of one of their great advantages as manufacturing sites. If a manufacturing plant cannot be established along the Arthur Kills, or a large city grow up there with the right to discharge sewage into the salt water; if investors in such enterprises must contemplate an enormous bond issue to take care of sewage, the manufacturing plant or the city might as well be located up in the mountains. Engineers will discriminate between drainage into salt water and drainage into a fresh-water stream, and, while the speaker agrees with Mr. Owen that the pollution of streams now used or likely to be used for water supply should be prohibited, the same arguments do not apply in the case of tidal streams; otherwise sewage purification works would be required for cities like New York and Brooklyn.

Mr. Maignen. P. A. Maignen, Esq.—In Paris the distribution of the water supply is double and of two qualities; raw river water for industrial purposes, street sprinkling, fire extinguishing and other base uses—sold at 12 cents per 1 000 galls.; and spring or assimilated (filtered) water for drinking and household purposes—sold at 24 cents per 1 000 galls.

The potable water distribution, being kept at a higher pressure than the industrial water, became largely used for hydraulic elevators and other purposes as motive power. In order to discourage this

misuse of the pure water, the rate for such purposes was raised to 48 cents per 1 000 galls. This differential rate has prevented the abuse of the pure water, so that filtration plants have to be provided only for water legitimately used and well paid for. Mr. Maignen.

Another reason for which French engineers advocate the double canalization system is to maintain an even pressure in the pure water distribution, so as to reach the highest points of the cities at all times. It is customary, in many towns, to water the streets and flush the sewers, for several hours every day. With a single canalization the upper parts of the city would be deprived of water by the simultaneous opening of the low-town hydrants; this trouble does not occur with a double canalization.

Filtration, as an art, is still in its infancy. It is a mistake to ask too much of a single filtering operation. The work should be divided. Something has already been accomplished in this direction by the use of storage reservoirs, but unless they are, as in London, large enough to render pumping during storms unnecessary, they are only extensions of the river, and of little use. If storage reservoirs are wanted in case of accidents to the pumps, by all means build them, but, in the speaker's opinion, clarification can be obtained equally well by quick upward filtration through coarse sand or other coarse material—the choice varying according to the particular quality of the water to be dealt with—and at less expense.

Sand filtration, as carried on in Europe, with sundry improvements, could be applied as a secondary operation. And it is possible to find, in the large industries, such as sugar and chemical works, breweries, wine producers and the like, which have recourse to filtration, some ideas which may be of service in carrying out a third and final operation in the purification of water for city supplies.

L. L. Tribus, M. Am. Soc. C. E. (by letter).—For twenty years past the providing of suitable water supplies for thickly settled districts has been receiving more and more intelligent consideration; the relation of physical health to the wealth and efficiency of a community has been more carefully studied from the economic as well as the æsthetic standpoint; and to the ends at issue have been brought the brains, energies and conscientious efforts of engineers, chemists and even of municipal officials themselves, who have been popularly classed as men devoid of other than political instincts. Mr. Tribus.

To a more limited degree, the disposal of the wastes of civilized life, passing off as sewage, has received careful attention, and is beginning to be treated as it deserves, as one of the most important subjects for the engineer to consider. It is not alone necessary to provide human beings with pure food and drink; it is equally necessary to remove the refuse after the valuable parts are used.

The questions presented for these informal discussions, relating to

Mr. Tribus. Stream Contamination by Sewage, Purification of Sewage, and Filtration of Water, are so correlated as to make their separation here scarcely wise, although certain features of each are distinct.

By far the larger number of cities (the term "cities" including all communities making use of municipal systems of water supply or sewerage), take their supply from rivers or lakes, larger or smaller as the case may be; those using the various forms of driven and artesian wells not, as a rule, having the classes of waters that need artificial purification. This being the case, it is evident that the source of supply should be maintained in all possible purity, rather than that the water should be permitted to deteriorate, and then need to be purified by some form of filtration, and at great expense.

Consequently, defiling matter, as certainly raw sewage must be considered, should not be admitted to a lake or stream if there is any possibility of injuring the water supply of another place. There is no justice in permitting one community to save itself trouble and expense by entailing trouble and expense on another, or in event of that other not having sufficient enterprise to protect itself, and perhaps furnishing to it elements of disease and danger. Rights to the waters of a stream have caused litigation for centuries, but it is only in comparatively recent times that "use" of the stream has been considered as requiring, morally, if not always legally, passing it on in unimpaired purity.

Certain classes of sicknesses known as "Filth" diseases, of which typhoid is the most prominent, are usually attributed, and probably with justice, to the dissemination by the water supply (or milk, directly associated, too often, with it) of germs contributed by sewage to the sources of the water system. Dilution of sewage with water has been largely looked upon, even by engineers, as a means of purification, and to some extent it is, but dilution does not destroy germs; and disease germs in a large volume of water have just so much more chance of dodging the other forms of bacterial life which seek to destroy them.

Not many years ago, purification of sewage was not an unmixed success, chiefly because of unintelligent design and carelessness in the operation of plants, the method of Nature's action not being entirely understood, or, if so, then ignored; the germ theory had scarcely been promulgated, and the existence of bacteria of different kinds, which, like people, preyed on each other, was unknown.

It would be a difficult matter to estimate in dollars and cents the losses of property, the destruction of valuable material, and the injury to lives, directly traceable to the emptying of crude sewage into streams; yet few will deny that the aggregate sum would be vast. The driving away or destruction of fish life, removing thereby a very valuable article of food; the depreciated value of property adjoining such sewage contaminated streams, and the actual deaths of hundreds or

thousands of human beings, from the use of polluted waters, sufficiently indicate the conditions at stake. The latter topic has been so admirably covered in recent papers on typhoid and allied diseases, with their relationship to water supplies, that merely speaking of the remarkable reduction in death rate, noticeable after the introduction of pure supplies, is all that is necessary here. Mr. Tribus.

The New York State Board of Health, in its policy of saving streams and lakes from further contamination, practically compelling cities to treat their sewage, seems to the writer to have taken the only proper action that justice and right can countenance; yet there are instances where such exclusion would appear an unnecessary precaution, for the rights of no one would seem to be injured by "direct" disposal.

It is a question, therefore, that had best be left to the intelligent and more enlightened sentiment and judgment of communities directly concerned, trusting to fully sufficient existing laws to prevent undue trespass on the rights of others. The writer believes, however, that the proportion of sewerage systems to be constructed in the future, without adequate disposal and purification plants, will be very small; and that a large number of the systems now in use will ultimately require treatment.

The same discussion applies very largely to the second question—"Should the Purification of the Sewage of Cities be Compulsory?" Present laws or court decisions cover fully all cases where non-treated sewage becomes a nuisance to residents of the city itself or to those of other places. Unfortunately, enforcement of laws seems to be a very difficult matter, and there is no reason to suppose that any new laws, even apparently more binding, would work to any greater advantage.

As far as applicability to large cities is concerned, new questions always arise. In almost all instances, large cities have grown up on the shores of tidal harbors, great lakes or large rivers, so that sewage flow bears usually so small a proportion to the volume of moving water, fresh or salt, that the dilution dissipates most of the cause of offence, though in the course of years, unmistakable and permanent indications of defilement can be noticed near the outlets of the sewers.

It is necessary to note clearly, what purification of sewage actually means. It is the total resolution into harmless inorganic elements, of all defiling or organic matters—a complete chemical combustion or oxidation. In this process, bacteria, abounding in the sewage itself, have a great deal to do; in the opinions of some, perhaps more than any other agent.

A direct answer as to its feasibility for large cities is impossible; each city must be judged by its own conditions. New York, Philadelphia and Chicago would illustrate the impossible, or at least, impracticable. Paris has, in a large measure, successfully embraced purification. London aims to make the effluent inoffensive, but it is

Mr. Tribus. far from being purified; its chemical treatment and subsidence being only an arresting of Nature's processes, and not a completion of them. To a thorough understanding of this last subject must those come who have the handling of sewage problems. Nature cannot be hurried, though she enjoys being assisted. We must conform to her laws or failure will ensue.

It is possible to purify sewage, and it is practicable, but it takes land in large areas to do it economically, and land properly selected and prepared, with the utmost care in its use, besides; or it takes water in such large volume that the same processes of nitrification and oxidation can be carried on without regard to time, and be conveyed away by currents or streams, so as to be inoffensive during the operations. The methods of treatment need not enter into this discussion; but it must be borne in mind that, unless local conditions absolutely prevent, some form of land treatment, without the use of chemicals, comes nearest to Nature's own most efficient process, and consequently is most likely to prove successful, even under severe climatic conditions.

Closely allied to the purification of sewage, and even more important to human life, is the providing of pure water supplies. Mountain lakes and brooks in uninhabited or sparsely settled districts can usually be taken as of sufficient purity, if the water be intelligently gathered and cared for.

The trend, however, of growth of population, is in the grouping of communities along the courses of the larger streams and on the shores of lakes. This entails more care in the selection of intake points, and is gradually tending to make undesirable the use of neighboring waters for public supplies without treatment before use. On the other hand, the smaller streams and lakes, not subject to pollution, do not furnish sufficient quantities of water for the larger cities, even when storm-waters, in addition to normal flow, are carefully conserved. It is not always feasible to secure ground-water or rock-well supplies, nor are such waters always safe or desirable themselves, so that recourse must be had in most cases to surface sources. Most persons will grant that a surface supply, from unimpeachable collecting grounds and stored in well-kept, clean reservoirs, is all that can well be desired, but in the more closely settled States, each year, such a condition is becoming much harder to find. Then, again, in the use of surface waters in their natural state there is always somewhat of an uncertainty, for, in spite of the most careful supervision of the water-shed, unless it be absolutely uninhabited, contaminating elements are always likely to appear.

The writer is not quite ready to insist that all public supplies be filtered, for here, as in sewerage questions, situation has a great deal to do with the method proper for adoption; but certainly, intelligent filtration never harms a water, while its lack may entail danger. Fil-

tration, in some form or other, undoubtedly began with man's use of water; its adoption, on strictly scientific lines for public use, is a development of very recent years. We may neglect consideration of the small household filters, the thousand and one devices that improve or injure the water according to the care in their use, and take up the broader field of filtration of municipal supplies as a whole, by the two general systems which may be classed as "Chemical" and "Natural." Mr. Tribus.

It is unnecessary here to discuss the comparative merits of either. It is sufficient to say that both have full fields for operation, the main point being that it is fully proven that impure waters can be filtered and rendered safe and desirable for domestic use. At the present day practically everything is measured by its value in dollars and cents, a materialistic theory which we have to recognize even while combating it as a principle, so that the *pro* and *con* of filtration for any particular city seems to be: Does the advantage of a purer water, in decreased death rate of population, figuring life at a minimum value, or a gain to the community of the work of so many persons annually, equal the operation, interest on cost and appropriations for a sinking fund for a filtration plant? Of course, taxes and politics, and various other, often conflicting, interests are also considered; though impaired health, more or less discomfort, and offensiveness to eye and nose, seem to have but little weight. Usually the activity of a violently deadly epidemic is necessary to impress the weight of the argument.

The writer is amply convinced that the coming solution of the public water question, considered as a whole, is in "purification," implying rather more than the narrower term "filtration," though that, in its growing ordinary acceptation, includes the very complex operations of Nature which result in thorough purification, hence safety for the health of, and satisfaction in other uses by, the public at large.

Palmer C. Ricketts, M. Am. Soc. C. E.—(by letter).—The great value and advantage of maintaining, if it were possible, all streams, large and small, in the purity they exhibited before there were settlements upon their banks is self-evident. The fact, however, remains that, to a greater or less extent, streams always have been and always will be natural sewers. They will be as long as water keeps flowing from the land into them and carrying with it impurities dangerous to health. Those flowing in thickly populated valleys certainly will be, even if the ordinary sewage of the cities on their banks be purified before being discharged into them. The fæcal matter of one inmate of one farm house may cause more havoc than a year's sewage flow from a large town. Mr. Ricketts.

As an example, consider the Mohawk River Valley in New York; narrow, with a long line of villages, towns and cities along its banks, and elsewhere quite thickly settled. If no sewage from towns and cities were permitted to be discharged into it the water would, never-

Mr. Ricketts. theless, be unsafe for drinking purposes, if not filtered. This river discharges into the Hudson River below the intake of the Troy waterworks. Below Troy, the cities of Albany, Hudson and Poughkeepsie filter the water pumped from the Hudson River. At least the last two have filtered it for years and the first will shortly have its extensive plant in operation. Under these circumstances it would be extremely difficult to force the great population of the towns and cities in the Mohawk Valley to purify their sewage before discharging it into the river, in order that its water might be more pure, but not pure enough to be drunk with safety.

In the "Report on the Chicago Drainage Canal by the Commission appointed by the Mayor of St. Louis, Mo., 1899," may be found the paragraph:

"By general law of the United States all cities must, sooner or later, be prevented from polluting the natural water courses of the country with sewage."

However much jurisdiction the United States may have in the Chicago Drainage Canal question, it would have to be considerably stretched to cover the case of a non-navigable stream, like the Mohawk, lying wholly within a single State and discharging into a river lying, as far as that part of it valuable for water-supply purposes is concerned, in the same State. And being thus practically a State question, how much chance is there that this large part of the population can, by act of Legislature, be put to great expense without proof that it is necessary.

When the writer became engineer of the Public Improvements Commission of the City of Troy, N. Y., and had, in consequence, to design a complete system of sewerage for the city, he was asked whether it would not be advisable to build an intercepting sewer along the river and purify the sewage of the whole city at the south end before discharging it into the Hudson River. This would have entailed pumping 9 000 000 galls. a day, exclusive of any rainfall, to a height of about 200 ft., purifying it, and discharging the effluent into the Hudson about 5 miles below the mouth of the sewage-laden Mohawk River. Twenty-five years ago the City of Troy began to pump most of its water supply from the Hudson above the mouth of the Mohawk, where the water was quite pure. Instead of doing this it could have obtained a bountiful supply by a gravity system at a reasonable cost. In the meantime the river has become more contaminated above the intake, by the sewage of towns which have since grown larger, and the quite pure water supply still exists in the highlands back of the city. Under these circumstances could Troy, in equity, compel the towns above to purify their sewage?

The writer has thus taken up the conditions existing in one neighborhood to show that no radical general rule can reasonably be made

regarding sewage purification. General laws, absolutely prohibiting the discharge of unpurified sewage into streams, would in many cases be destructive and unjust; and, in the design of a system of sewerage, it is well to remember that it is easy to put a community to unnecessary expense to meet possibilities which never materialize, and sometimes to prevent the construction of much-needed works by prohibitive and unnecessary conditions. Mr. Ricketts.

On the other hand, it is not necessary to state that many cases exist where the purification of sewage before its discharge into streams is advisable and necessary. This, of course, is particularly the case where the streams are small and the country thickly settled. Those in the neighborhood of Boston afford a good example. It also seems evident that even if a stream is not to be used for a water supply it should not be permitted to become a nuisance because of sewage discharged into it.

As to the pure-water question for cities, the writer believes filtration to be its coming solution. More or less pure water may be used without filtering if it comes from a source uncontaminated by the sewage of villages or cities, but the only way to be sure that it shall be reasonably pure is to filter it.

James Smith Haring, M. Am. Soc. C. E. (by letter).—There is a phase of the discharge of raw sewage into rivers and streams which was not mentioned in the oral discussion of this subject, that of the minimum flow, in either rivers or streams. Mr. Haring.

This is a matter of great importance in considering the question of permitting untreated sewage to enter sources of water supply, not only as regards the effect upon the purity of the supply for other localities, but also the relation which the amount of sewage passing into the stream has to the quantity of water flowing therein, to prevent the nuisance of the collection of undiluted sewage in the immediate vicinity.

Most western rivers are subject to large variations in both depth of flow and area covered during the periods of maximum and minimum seasons. Although the volume of water is calculated from the total quantity passing in the channel, yet it may be so divided and diverted as to leave the conditions attending the dilution of sewage quite at variance with the permissible allowance. Especially is this true of rivers having very rocky channels, in which the flow is concentrated at certain points into small cross-sections; while, at other places, the water is spread out over a very shallow but wide and stony bed and almost disappears. This is also true of a river of sandbars and marshes. The collection of sewage in beds of this character certainly emphasizes the necessity of compelling its treatment before discharging it into streams used for water supplies. It seems quite unnecessary to require any treatment of sewage before disposing of it in tidal

Mr. Haring. salt-water bodies. Cases might arise where sewage might be caught on a flood tide and carried back upon the shores and cause a nuisance, but the conditions would not be general, and the writer does not think that in the discussion of this subject such a contingency was anticipated.

The only way to make the purification of water supplies effective is to first prevent, by Federal laws, the pollution of rivers and streams. Too many of our rivers are in the jurisdiction of more than one State to make the State control of these matters effective, and the personal feelings of lawmakers are too largely acted upon by the fear of giving offense to some person or interest in their constituency to make a success of individual State legislation. Then, too, individual cities and towns will procrastinate to a degree almost, if not quite, criminal, rather than press on to a reform, or even to provide the means necessary to remedy or prevent the pollution of streams, because of the attending expense; and though conscious and confessing the, shall we call it, crime, permit it to go on. Only a general law, irrespective of State or city, can prevent effectually what will be in a few years, if it is not in some places already, an evil of great magnitude.

Filtration is destined to be the important agent to prevent wholesale contagion from water-borne diseases, even though it be compulsory by law to treat sewage before permitting it to flow into streams, and even though the effluent be purified "to a drinking-water standard," as one business concern engaged in that work now claims to do (though the standard may need qualification). The rapid development in the education of the people demands the filter as a guaranty of the best that can be had in water supply. Though sewage may be treated and made inoffensive by the expenditure of money, yet water supplies cannot be manufactured, and the daily increasing news of water famine all over the country admonishes us of such conditions.

Mr. Fuertes. JAMES H. FUERTES, M. Am. Soc. C. E. (by letter).—A brief analysis of what is already published on these subjects will be, it seems to the writer, of more value than the addition of new compilation to obscure further the simple principles governing the points under discussion.

The right of polluting streams, under certain conditions and to a certain extent, has been a large factor in the progress of all advanced countries. In some of our States the Mill Acts, passed to foster growing industries, would be inoperative if this right were not conceded. The term pollution is always comparative. No water found in Nature is pure, the purest being that from mountain springs. Going down the scale in the order of comparative pollution, would then be classed ordinary ground-waters, waters impounded in large storage reservoirs, waters of large normal rivers, waters of large lakes taken some distance from the shores, waters of upland streams, and, lastly, wells and bodies of water polluted directly by sewage matter near the point of abstraction.

Prevention of pollution must, therefore, imply degree of preven- Mr. Fuertes.
tion desirable, because all streams are already polluted by natural agencies beyond the control of man's legislation, unless flies and birds and winds and migratory animals can be legislated out of existence.

To prevent stream pollution absolutely would be undesirable. If a stream is not to be used for a water supply, or if its waters are not to enter into the composition of matters intended for human consumption, sufficient pollution prevention to insure its not being offensive in the slightest degree, or injurious to manufacturing interests, would be sufficient. This can be secured under the existing laws. If the water is to be used for a water supply, its protection must be more complete; but here, again, the laws are ample in scope. Not only would it be undesirable, but it would be impossible to secure the passage of absolute pollution-prevention legislation, because such laws would paralyze the manufacturing interests of the whole country, and would thereby arouse irresistible organized opposition. These views are expressed by the best expert opinion and by legislative bodies.

The discharge of a large amount of sewage into a small stream will, undoubtedly, make it offensive and unfit to drink; but the discharge of a much greater amount into the Upper Mississippi River, would be undetectable at its mouth. Thus, at New Orleans, the effect of the sewage discharged into it and its tributaries by all the cities between the Alleghenies and the Rockies is completely masked by the much greater relative pollution of the same kind, caused by surface washings from the water-sheds of these streams. In this connection, it will be interesting to observe whether or not the influence of the Chicago sewage, after it is directed into the Mississippi River, can be detected at St. Louis.

As a health protective measure, for cities on large rivers like the Mississippi, Ohio, Hudson, etc., it would be more efficacious and less expensive for all to discharge their crude sewage into the rivers and then purify their drinking water, than for all to purify their sewage and not their water supplies. On smaller streams both processes may be necessary. So far as the prevention of stream pollution is concerned, the laws are adequate; witness the recent cases of Danbury, Conn.; New Britain, Conn.; Santa Rosa, Cal., and numerous others.

The writer, in a statistical study compiled in 1896,* called attention to the extent and importance of pollution from surface washings, and since then has had verification of his deductions by seeing the same views expressed in the findings of a special investigation for the War

* "Water and Public Health. The Relative Purity of Waters from Different Sources."

Mr. Fuertes. Department on the cause of typhoid fever in the army camps during the late war, and in the report of the board of experts appointed to report on the improvement of the water supply of the City of Philadelphia.

The indications are that, with a little more popular education in these matters, the people will, before many years, demand the purification of all waters obtained from surface gathering grounds, rivers or lakes.

The purification of the sewage of large cities is not always practicable. So far as safe-guarding public health is concerned, it is vastly more important to sterilize all the air we breathe than to purify the sewage—for instance, of New York City. One would be just about as feasible, from a financial point of view, as the other. Probably there are, at least, one hundred times more deaths in the city resulting from breathing impure air than can possibly be traced to the combined effect of the sewage of the city and the infection of water and food supplies.

Filtration, if properly planned and conducted, is the best and most satisfactory way to purify drinking water, so far as we know, to-day. What the next fifty years will bring forth, however, is beyond the power of any mortal to prophesy. In the past two decades, so many of our standards have been changed, and so many useful scientific inventions have been perfected, that great advances are to be expected in every field.

There may be individuals in the next generation who will live to see 10 000 000 people resident in Greater New York. When that time arrives the water needed by the population will equal the present low-water flow of the Hudson River at Albany—draining over 8 000 square miles of country. The writer does not believe that any one now living can say how that water will be derived.

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No. 858.

WHEEL CONCENTRATIONS AND FATIGUE FORMULAS IN BRIDGE DESIGN.

An Informal Discussion at the Annual Convention, June 27th, 1899.

SUBJECTS FOR DISCUSSION:

"Should the use of the Method of Wheel Concentrations be discontinued in determining the Stresses in Railroad Bridges?"

"In view of present knowledge of the Effect of Repeated Applications of Load, should Fatigue Formulas be used in Bridge Design?"

By Messrs. GUSTAV LINDENTHAL, PALMER C. RICKETTS, W. A. DOANE, GEORGE S. MORISON, W. L. COWLES, E. M. SCOFIELD, HENRY B. SEAMAN, J. P. SNOW, EDWIN THACHER, J. A. L. WADDELL, O. E. SELBY, HENRY S. PRICHARD, CHARLES B. WING and EDGAR MARBURG.

GUSTAV LINDENTHAL, M. Am. Soc. C. E.—The objection to the method of wheel concentrations for finding the strains in bridge members does not arise so much from the greater labor of calculation, but because the assumption of wheel loads as resting on knife-edges is a false and unnecessary refinement, for, in any case, the rails and ties intervening between the wheels and the bridge structure distribute the concentrated loads upon the length of the wheel base. It may be correctly assumed, for the bridge structure, that the loads rest on skids, instead of on wheels; the rails representing the skids. The strains in the floor and in the trusses are thus more accurately found, for the train load plus the excess load of the locomotive, than by the method of wheel concentrations, which should be discontinued. Mr. Lindenthal.

Mr. Lindenthal. There are indications that the growth of weight in locomotives will soon come to an end; it will be determined, not by the strength of rails, bridges or drawbar heads, which can be increased at will, but by the hardness of steel, in wheel tires and rails, necessary to resist the wear and tear of the heavy engines. It will reach the limit when steel in the railhead would be crushed, or, if made still harder, would be too brittle for safety. It may be fairly assumed that 120 tons on a wheel base of 15 ft. is the practicable and economic limit of weight concentration on railroad track. This represents a quiescent load of 16 000 lbs. per lineal foot, corresponding to 30 000 lbs. per wheel; or from 60 000 to 75 000 lbs. pressure per square inch on the wheel tire and rail, which is close to, or exceeding, the elastic limit of hard steel.

Some rails have, indeed, been made of hard nickel steel, which may safely resist greater pressures, but they cost about double as much as ordinary rails. They are used only for the outside of curves, and, therefore, do not determine ordinary service conditions.

Fatigue formulas in bridge designing made their first appearance in American bridge specifications about fifteen years ago, mostly in the form of the Launhardt and the Launhardt-Weyrauch formulas, or modifications thereof. The previous practice, which is also still largely followed (and sometimes combined with the fatigue formulas) consisted in the variations of the unit stresses for different bridge members, depending upon their position in the bridge, using the lowest unit stresses in the floor system and the highest in the chord members of the trusses. This was done according to the judgment of the designing engineers, out of which gradually had grown certain standards. The exploitation of the fatigue formulas was probably prompted by a desire to bring the arbitrary but well-founded standards, as regards unit stress, under a uniform scientific rule, although that expectation was not fulfilled.

As every one knows, the formulas were deduced by Professors Launhardt and Weyrauch, who were not themselves the experimenters, from certain tests for the Prussian Government, made by Wöhler and Spangenberg about thirty years ago, with steel and iron bars, which were subjected to rapid repetitions and reversions of stress.

The formulas were regarded at the time as examples of elegant mathematical reasoning, and their plausibility won many believers. Nevertheless, their application to bridge designing is absurd, because the conditions on which they are based do not obtain in bridges. This is well recognized by European engineers, who rarely, if ever, use them.

Wöhler strained his iron and steel close up to and over the elastic limit at the rate of 5 to 20 repetitions per second. These conditions may sometimes occur in machinery, car axles, connecting rods, rails, and other like structures, where the tensional and compressional strains are nearly always the effect of sudden and long-continued inter-

mittent forces, and complicated with bending, shearing, or torsional strains. But, even for them, the correctness of the formulas is open to serious question when the stresses are kept far below the elastic limit. Mr. Lindenthal.

Steel rails, made of 80 000-lb. steel, for instance, and subject to fiber stresses from + 27 000 to — 27 000, and higher, from fast and frequent trains, are found to be perfectly safe. They wear out, but they do not break down.

The strains in iron and steel bridges, however, are of an entirely different character. They are primarily static, and in well-designed bridges rarely exceed half the elastic limit of the material. They are not complicated with other strains, as in machinery designing; and, moreover, the maximum strains occur only at long and rare intervals, and in cumulative form.

It surely is not proven, that slow repetitions and reversions of stress, far within the elastic limit of the steel and iron, affect in any way their durability. There is not a single authentic fact or experiment known to support such a contention. Wöhler, who was well qualified to scientifically interpret his own tests, never made such a claim himself.

The values of the Launhardt-Weyrauch formulas agree with Wöhler's tests only partially, within certain limitations of stress. But there are no facts and no tests to prove their applicability to stresses one-half or one-third as low as those which Wöhler used. In the nature of things, such experiments cannot be made. How many thousand years would it take to subject a steel bar of 1 sq. in. to reversions of stress from 12 000 lbs. in tension to 8 000 lbs. in compression, at the rate of one reversion per minute, and at intervals of one hour, before it would suffer in strength, if it ever would? This is assuming, indeed, a higher strain, a quicker reversion and a shorter interval than actually occur in the chords of many existing cantilever and arch bridges, dimensioned according to fatigue formulas. The waste of material which they imply, particularly for members subject to reversion of stress, is not very important for ordinary spans; but for long-span bridges, where the maximum strains occur only under extraordinary and rare conditions, it is a very serious matter, as the bridge is simply loaded down with useless material, adding to its cost, but not to its strength or durability. In the speaker's practice he has never used them, being convinced that the old English practice, also endorsed by Rankine, of separating the strains from dead and live load, and determining the cross-section for both, by using larger unit stresses for the former, and smaller unit stresses for the live load, and then adding the results, is founded on the most rational basis thus far conceived. With the great variations of load conditions and material in steel bridges, it is more than doubtful if the judgment, in that respect, of the experienced designer can ever be replaced in practice by any one or more formulas.

Mr. Ricketts. PALMER C. RICKETTS, M. Am. Soc. C. E.—Some years ago, in discussing the paper "Some Disputed Points in Railway Bridge Designing,"* the speaker predicted that the method of wheel concentrations would be abandoned in a very few years. It is passing into disuse, and that it has not disappeared long ago is to be attributed rather to the conservatism than to the common sense of bridge engineers. It should be abandoned, not because the calculations consume more time and are more difficult than by other methods, but because the method is unscientific, and its accuracy is only apparent. It is irrational and unscientific to obtain stresses by carrying to such extremes calculations which take no account of dynamic effects, while the actual errors in design due to ignorance of dynamic effects are probably many times greater than those due to differences in methods of obtaining stresses. In other words, such calculations will be unnecessary refinements as long as we have to guess at allowed unit stresses.

The subject of fatigue formulas and their use in bridge design was so fully covered in the paper "The Launhardt Formula, and Railroad Bridge Specifications,"† that it is almost unnecessary to discuss it now. About 80% of those who took part in that discussion were of the opinion that fatigue formulas were wholly irrational, and for that reason should not be used.

Mr. Doane. W. A. DOANE, M. Am. Soc. C. E.—The stresses in the various parts of railway bridges can be more easily calculated, and with fully as close an approach to the strains which will actually exist in the structure after completion, by using the method of uniform live loads, rather than by taking certain live loads, which may not obtain with one locomotive out of ten.

The speaker is, therefore, in favor of assuming uniform loads in bridge design, the more so, perhaps, since the general advent of 80-lb. or 90-lb. rails helps materially to distribute the wheel loads on the track stringers, or on the plate floor, as the case may be.

Mr. Morison. GEORGE S. MORISON, Past-President, Am. Soc. C. E.—There is one idea which the speaker has thought over for a good many years. He has always felt that the use of locomotive wheel-base concentration was radically wrong. That it will give results which are not materially bad is certainly true, but the principle of it seems to be very far from what should be followed. It is, briefly, that we take a piece of rolling stock of special weights at special distances and make that a basis for a structure which we expect to last 50 years, and which probably will last 25; and yet that same locomotive will be superseded in $2\frac{1}{2}$ years.

* By J. A. L. Waddell, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 77.

† By Henry B. Seaman, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xli, p. 140.

Mr. Morison.

The original idea has been somewhat improved by using typical locomotives. The specifications of 20 or 25 years ago contained locomotives with the weight on the wheels carried down to the nearest 10 lbs., and the distances between wheels given to the nearest inch—the speaker thinks that some of them went to fractions, but trusts that this was not really the case. The speaker has felt that this was entirely wrong, and a good many years ago he adopted a method which he has always used in drawing his specifications, and which he still thinks gives better results than any system of wheel-base loads.

There is one feature of the wheel-base load which, if drawn out on a diagram, shows its defects. It does not give a uniform line gradually progressing for the varying length of spans, etc., but a line of slight undulations. Now, those undulations ought, in some way, to be eliminated, and something should be obtained which would give a gradual rate of progression from a very short span up to a span of such length that a uniform load can be taken throughout. There are really four different things to be provided in selecting the loads for which railroad bridges are to be proportioned. They are: (1) a heavy wheel-base load; (2) a moderate load for greater lengths; (3) a gradual transition from the heavy wheel-base load down (if measured by weight) or up (if measured by length) to the lighter uniform load; (4) provision for an excess in those members most affected by moving loads.

To these, perhaps, one other should be added. It is simply the provision that for extremely short spans, those less than wheel-base loads, provision should be made for an excessive concentration, and that load ought to be very excessive; it has been the speaker's practice to consider that it was possible that the whole 20 wheel-base loads could be concentrated on two axles, or, perhaps more properly, that half of it could be concentrated on one axle.

It has also seemed expedient to have these provisions governed by coefficients rather than by absolute amounts, and to adapt different structures to loading of different weights, by varying the unit load to which the coefficients are applied.

In other words, there should be a table of coefficients, or a rule for determining the coefficient, for each length of span. Bridges should be classified for the duty they have to perform, and, by applying those coefficients to the unit weight, the weight which is to be used in calculating every length of span could be ascertained at once.

Following this general idea, the speaker adopted, and this was done about 1882, 20 ft. as the wheel-base length and 120 ft. as the length of two locomotives. He then adopted, as the uniform load for 120 ft. and all greater lengths, a weight per lineal foot varying from 2 000 to 4 000 lbs., according to the class of the structure. On the wheel-base length of 20 ft. this unit load was doubled. The transition was effected by adding 1% to the uniform load for each foot of length

Mr. Morison. of structure less than 120 ft., this increase becoming 100% at the wheel-base length of 20 ft. Impact and irregular strains must be provided for chiefly in those members in which variations are rapid and reversals are liable to occur, that is, in the counter members of the web, and to a varying extent in the main members of the web. This extra provision was then made by considering what portion of the strain in any web member would exist under a load equal in intensity to the moving load, but uniform throughout the span. Deducting this from the maximum strain in such member left a remainder which varied from nothing at the end of the span to the full counter strain at the center, and which might be called the variable effect. This variable effect was then estimated on a basis of $1\frac{1}{2}$ times the regular moving load, and gave results which, from their uniform increase from the ends to the center and their general character, seemed to conform with the proper ideas of proportioning a structure.

On this basis, nearly every bridge the speaker has built for the last 16 years has been designed. There are, however, some defects. If applied on the basis of the length of the span, the load for the shearing strains is too low; if applied on the basis of half the length of the

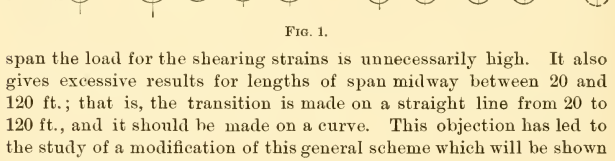

FIG. 1.

span the load for the shearing strains is unnecessarily high. It also gives excessive results for lengths of span midway between 20 and 120 ft.; that is, the transition is made on a straight line from 20 to 120 ft., and it should be made on a curve. This objection has led to the study of a modification of this general scheme which will be shown in the following pages.

The calculations as given are applied to only one unit weight of train, and this has been taken at 5 000 lbs. per lineal foot, which is believed to be the highest unit weight now in use. The result, however, as obtained, is a series of coefficients which can be applied to any unit weight, which unit weight would probably, under present practice, be made to vary from 3 000 to 5 000 lbs.

Take as an illustration of a standard typical locomotive Fig. 1, used in a recent paper in the *Transactions* of this Society.* This engine measures 57 ft. between corresponding points, when coupled up in a train of locomotives. The total weight is 276 000 lbs. This weight has been materially exceeded by recent engines, the two heaviest reported being a consolidation pusher on the Lehigh Valley Railroad,

Transactions, Am. Soc. C. E., Vol. xli, p. 140.

and a consolidation engine running on the Union Railway of Pitts- Mr. Morison.
burg, full descriptions of both of which have been given in several of the technical papers. The length is somewhat less than that of the modern consolidation engine, which is usually somewhat over 60 ft., the total length of the Lehigh Valley engine being 62 ft., and that of the Union Railway engine exceeding 65 ft.

For purposes of simplicity it would seem advisable to adopt an engine 60 ft. long and to avoid the odd weight on the truck wheels and irregular distances between wheel bearings. These conditions are met by the engine shown in Fig. 2.

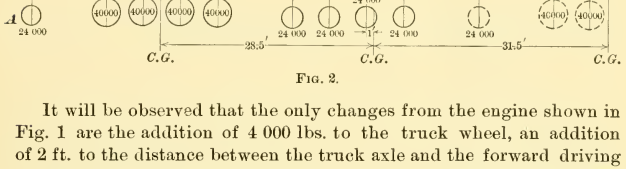

FIG. 2.

It will be observed that the only changes from the engine shown in Fig. 1 are the addition of 4 000 lbs. to the truck wheel, an addition of 2 ft. to the distance between the truck axle and the forward driving axle and the reduction of 1 ft. between the two central tender axles. This engine is 60 ft. long. It weighs 280 000 lbs., 4 000 lbs. more than the other. Every one of the weights is divisible by 4 000 and every distance is a multiple of 5; and yet the strains from this engine would differ so little from those of the engine shown in Fig. 1 that they would not be noticed in any structure that would be built.

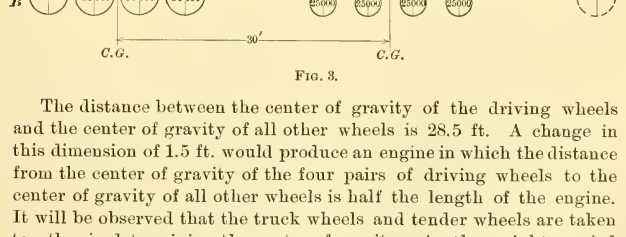

FIG. 3.

The distance between the center of gravity of the driving wheels and the center of gravity of all other wheels is 28.5 ft. A change in this dimension of 1.5 ft. would produce an engine in which the distance from the center of gravity of the four pairs of driving wheels to the center of gravity of all other wheels is half the length of the engine. It will be observed that the truck wheels and tender wheels are taken together in determining the center of gravity. As the weight carried on the truck is largely a matter of adjustment, it seems wise to eliminate the truck wheels in a typical engine and to add their weight to

Mr. Morison. the driving wheels. If these changes are made and the total weight of the engine is increased to 300 000 lbs., or 5 000 lbs. per foot, the result is the typical engine shown in Fig. 3. This engine does not differ materially from that shown in Fig. 4 in any of its effects on a bridge.

Fig. 3 is an exceedingly simple engine so far as its weight on the tracks is concerned. On each of the four driving axles is a weight of 50 000 lbs. On each of the four tender axles is a weight of 25 000 lbs., or just half that of the driving axles; the center of gravity of one set of wheels is just 30 ft. from the center of gravity of the other set of wheels, and every distance is a multiple of 5.

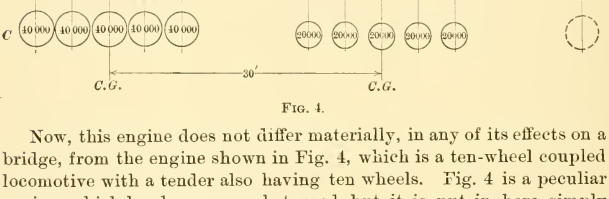

Fig. 4.

Now, this engine does not differ materially, in any of its effects on a bridge, from the engine shown in Fig. 4, which is a ten-wheel coupled locomotive with a tender also having ten wheels. Fig. 4 is a peculiar engine, which has been somewhat used, but it is put in here simply because, though the nominal wheel base is a little more, the actual wheel base is practically the same, and in the effects of the two engines on a bridge hardly any difference can be found.

These engines do not represent any now running, but are typical engines for use in calculation, and producing more excessive effects

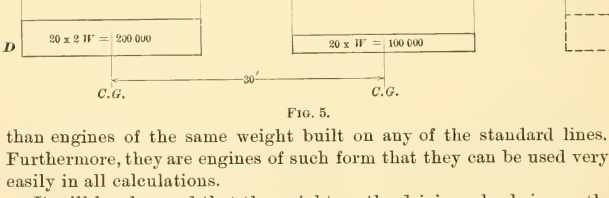

Fig. 5.

than engines of the same weight built on any of the standard lines. Furthermore, they are engines of such form that they can be used very easily in all calculations.

It will be observed that the weight on the driving wheels is exactly double that on the other wheels, that the two sets of wheels are arranged in precisely the same way around their centers of gravity, and that the distance between the two centers of gravity, is half the length of the locomotive, or 30 ft.

If we substitute for the specially applied wheel load, a uniform

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FIG. 6.

Mr. Morison. load of equal intensity, of a length equal to the number of wheels multiplied by the distance between their centers, we have the results shown in Fig. 5.

This loading will also represent passenger locomotives of rather extraordinary dimensions. For instance, the engine wheel-base load of 200 000 may be considered as representing a locomotive with four driving wheels and 50 000 lbs. on each driving wheel; while it is to be hoped that no such locomotive will ever be built, pressures of approximately 50 000 lbs. per wheel now exist in badly counterbalanced engines when running at excessive speeds. That is a thing which bridge specifications have never taken into consideration, and which throws all the ordinary refinements of wheel-base loads out of gear. These strains, however, do not exist on all four wheels at the same time; their influence is felt on single stringers of short lengths, but not on complete spans.

In order to show how closely this typical loading agrees with the several locomotives, Fig. 6 has been prepared, and shows the bending moments under the several conditions. This drawing is taken on the basis of a 60-ft. span with the center of the driving-wheel base placed at the center of the span. The moments of the five different engines and loadings which we have been considering are drawn to scale.

Taking a moving load composed of a continuous train of the typical "*D*" loading (Fig. 5) as a basis of calculation, Table No. 1 shows the maximum shearing strains at the end of the several spans named under such moving load, the lengths of span varying by 20 ft.

TABLE No. 1.

Length of span. Feet.	*a* End shear by typical load.	*b* End shear by uniform load (5 000 lbs. per linear foot).	$a \div b$ Coefficients.
20	100 000	50 000	2.000
40	156 250	100 000	1.563
60	200 000	150 000	1.333
80	250 000	200 000	1.250
100	302 500	250 000	1.210
120	350 000	300 000	1.167
140	400 000	350 000	1.143
160	451 563	400 000	1.129
180	500 000	450 000	1.111
200	550 000	500 000	1.100
220	601 136	550 000	1.093
240	650 000	600 000	1.083
260	700 000	650 000	1.077
280	750 893	700 000	1.073
300	800 000	750 000	1.067
320	850 000	800 000	1.063
340	900 735	850 000	1.059
360	950 000	900 000	1.056
380	1 000 000	950 000	1.053
400	1 050 625	1 000 000	1.051
420	1 100 000	1 050 000	1.048
440	1 150 000	1 100 000	1.045
460	1 200 543	1 150 000	1.044
480	1 250 000	1 200 000	1.042
500	1 300 000	1 250 000	1.040

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In the same way the bending moments in the center of the span are given in Table No. 2.

TABLE No. 2.

Length of Span. Feet.	a Moment by typical load.	b Moment by uniform load (5 000 lbs. per linear foot).	$a \div b$ Coefficients.
20	500 000	250 000	2.000
40	1 500 000	1 000 000	1.500
60	2 750 000	2 250 000	1.222
80	4 500 000	4 000 000	1.125
100	6 500 000	6 250 000	1.040
120	9 000 000	9 000 000	1.000
140	12 500 000	12 250 000	1.025
160	16 500 000	16 000 000	1.031
180	20 750 000	20 250 000	1.025
200	25 500 000	25 000 000	1.020
220	30 500 000	30 250 000	1.008
240	36 000 000	36 000 000	1.000
260	42 500 000	42 250 000	1.006
280	49 500 000	49 000 000	1.010
300	56 750 000	56 250 000	1.009
320	64 500 000	64 000 000	1.008
340	72 500 000	72 250 000	1.003
360	81 000 000	81 000 000	1.000
380	90 500 000	90 250 000	1.003
400	100 500 000	100 000 000	1.005
420	110 750 000	110 250 000	1.0045
440	121 500 000	121 000 000	1.004
460	132 500 000	132 250 000	1.002
480	144 000 000	144 000 000	1.000
500	156 500 000	156 250 000	1.002

These calculations are based on a train consisting entirely of locomotives, and such trains are not run. Approximations to such trains are found, however, in mineral traffic, in cars of special character. A car designed and built by the Goodwin Patent Gravity Dumping Car Company weighs 42 000 lbs., and is designed to carry 125 000 lbs. of ore, making a total weight of 167 000 lbs. on a length a trifle less than 36 ft., or very nearly 5 000 lbs. per foot. In mineral districts it would seem expedient to proportion bridges for weights equivalent to trains composed entirely of locomotives. In other districts it may be expedient to provide for only two locomotives and to reduce the weight of the following train to something less than the locomotive load. If this reduction is arbitrarily taken at 25%, the train load per foot will be three-quarters of the locomotive load per foot. It seems right to apply this correction in the form of a reduction from a uniform load rather than to make the locomotive an addition to it. On this basis Tables Nos. 3 and 4 have been calculated, Table No. 3 showing the maximum shearing strain at the ends and Table No. 4 the maximum bending moments at the centers of spans, caused by a train consisting of two locomotives followed by a uniform load of three-quarters of the average weight of the locomotives. It will be observed that these two tables are identical with the foregoing tables for all lengths under 120 ft.

Mr. Morison.

TABLE No. 3.

Length of span. Feet.	*a* End shear by typical load.	*b* End shear by uniform load (5 000 lbs. per lineal foot).	$a \div b$ Coefficients.
20	100 000	50 000	2,000
40	156 250	100 000	1,563
60	200 000	150 000	1,333
80	250 000	200 000	1,250
100	302 500	250 000	1,210
120	350 200	300 000	1,168
140	394 075	350 000	1,126
160	436 230	400 000	1,090
180	477 345	450 000	1,061
200	517 734	500 000	1,035
220	557 590	550 000	1,014
240	596 860	600 000	0,994
260	636 238	650 000	0,979
280	675 165	700 000	0,964
300	713 905	750 000	0,952
320	752 500	800 000	0,941
340	791 000	850 000	0,931
360	829 400	900 000	0,921
380	867 555	950 000	0,913
400	905 740	1 000 000	0,906
420	943 860	1 050 000	0,899
440	981 925	1 100 000	0,893
460	1 019 940	1 150 000	0,889
480	1 057 910	1 200 000	0,882
500	1 095 840	1 250 000	0,877

TABLE No. 4.

Length of span. Feet.	*a* Moment by typical load.	*b* Moment by uniform load (5 000 lbs. per lineal foot).	$a \div b$ Coefficients.
20	500 000	250 000	2,000
40	1 500 000	1 000 000	1,500
60	2 750 000	2 250 000	1,222
80	4 500 000	4 000 000	1,125
100	6 523 437	6 250 000	1,044
120	9 037 500	9 000 000	1,004
140	12 260 187	12 250 000	1,001
160	15 718 750	16 000 000	0,982
180	19 413 437	20 250 000	0,959
200	23 336 250	25 000 000	0,933
220	27 490 234	30 250 000	0,909
240	31 875 000	36 000 000	0,885
260	36 240 230	42 250 000	0,858
280	41 583 435	49 000 000	0,849
300	46 837 500	56 250 000	0,833
320	52 500 000	64 000 000	0,820
340	58 687 500	72 250 000	0,812
360	65 250 000	81 000 000	0,805
380	72 187 500	90 250 000	0,800
400	79 500 000	100 000 000	0,795
420	87 185 500	110 250 000	0,791
440	95 250 000	121 000 000	0,787
460	103 687 500	132 250 000	0,784
480	112 500 000	144 000 000	0,781
500	121 687 500	156 250 000	0,779

The coefficients of the four tables have been plotted in Fig. 7.

Mr. Morison.

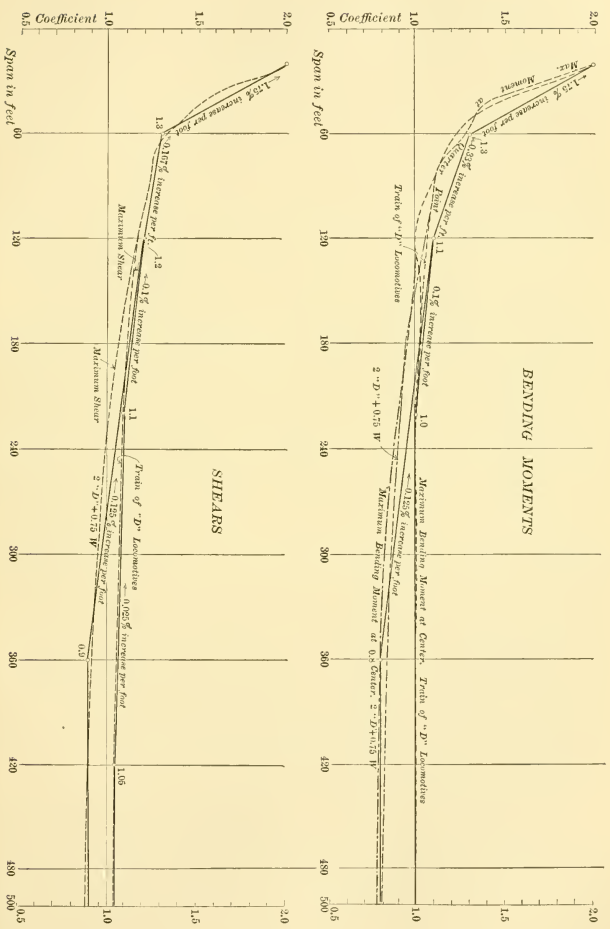


FIG. 7.

Mr. Morison. If these coefficients have to be calculated for every position of loading, there is no advantage in their use; furthermore, the coefficients of Table No. 2 show a series of maxima and minima which ought to be eliminated. To obviate these difficulties a few simple rules are proposed for fixing coefficients for every length of span; the coefficients, as determined by these rules will, in almost all cases, be slightly in excess of the calculated coefficients. The coefficients, as determined by these rules, are plotted as full lines in Fig. 7.

Rule 1.—Bending Moments. Uniform Locomotive Load.—Coefficient for 220 ft., and all greater spans, 1. Increase this coefficient by one-tenth of 1% (0.001), for each foot of decrease of length down to 120 ft., making the coefficient for 120 ft., 1.10. Increase this by one-third of 1% (0.0033), for each foot down to 60 ft., making the coefficient for 60 ft., 1.3. Increase by 1.75% (0.0175), for each foot down to 20 ft., making the coefficient for 20 ft., 2.

Rule 2.—Bending Moments. Two Locomotives Followed by Train.—Coefficient for 360 ft. and all greater spans, 0.8. Increase this coefficient by one-eighth of 1% (0.00125), for each foot of decrease of length down to 120 ft., making the coefficient for 120 ft., 1.1. Increase by one-third of 1% (0.0033), for each foot down to 60 ft., making the coefficient for 60 ft., 1.3. Increase by 1.75% (0.0175), for each foot down to 20 ft., making the coefficient for 20 ft., 2.

Rule 3.—Shearing Strains. Uniform Locomotive Load.—Coefficient for 420 ft. and all greater spans, 1.05. Increase this coefficient by one-fortieth of 1% (0.00025), for each foot of decrease of length down to 220 ft., making the coefficient for 220 ft., 1.1. Increase by one-tenth of 1% (0.001), for each foot down to 120 ft., making the coefficient for 120 ft., 1.2. Increase by one-sixth of 1% (0.00167), for each foot down to 60 ft., making the coefficient for 60 ft., 1.3. Increase by 1.75% (0.0175), for each foot down to 20 ft., making the coefficient for 20 ft., 2.

Rule 4.—Shearing Strains. Two Locomotives in Train Load.—Coefficient for 360 ft. and all greater spans, 0.9. Increase this coefficient by one-eighth of 1% (0.00125), for each foot of decrease of length down to 120 ft., making the coefficient for 120 ft., 1.2. Increase by one-sixth of 1% (0.00167), for each foot down to 60 ft., making the coefficient for 60 ft., 1.3. Increase by 1.75% (0.0175), for each foot down to 20 ft., making the coefficient for 20 ft., 2.

These rules are designed principally as a means of carrying easily and in compact shape these coefficients so that they can be handled quickly, without tabulation. A better method, however, would be to draw a curve governed by the different points which are used in the calculations, and then determine the coefficient for each foot of length from that curve. The speaker has endeavored to find some curve, with a simple formula, which might be used in this way, but, so far, he has not succeeded. Perhaps some other member of the Society may be more fortunate.

W. L. Cowles, M. Am. Soc. C. E.—The admirable paper entitled "General Criterion for Position of Loads causing Maximum Stress in any Member of a Bridge Truss,*" presented by L. M. Hoskins, Esq., affords an interesting mathematical treatment of the problem, and is a valuable contribution to the theoretical literature on this subject. Mr. Cowles.

The speaker desires, however, at the present time, to discuss briefly the necessity for finding this position, rather than the method of accomplishing the object.

Such a discussion naturally merges with the discussion of concentrated *versus* uniform loads, for the main object to be finally attained is the determination of the maximum stresses produced in the members of a truss by the application of a load within a satisfactory degree of accuracy and with the least labor on the part of the computer.

It must be conceded that a bridge is rarely loaded with the exact load for which it is designed, and in fixing upon the load to be used in making the calculations it is the object of the engineer to specify such a load, either uniform or composed of concentrated wheel-loads, as will produce stresses at least equal to, and preferably somewhat greater than will ever be reached in common practice, making due allowance for any probable increase in loading in the future. This being the case, it follows that extreme accuracy in the calculations is not essential, and, given a satisfactory degree thereof, the problem reduces to a determination of the simplest means of arriving at the results sought.

For ease and quickness of computation it is beyond doubt that a uniform load is preferable, and the speaker is in favor of the adoption of such loads, if railroad engineers can be induced to specify them when requesting propositions for bridge work. Notwithstanding, however, the extended discussion of this subject by the members of this Society, and the offers which have been made of tables indicating equivalent loads, a comparison of the bridge specifications of the railroads of the United States proves that, with a very few exceptions, the engineers of these roads still prefer to specify a loading consisting of a series of concentrated loads representing typical locomotives followed by a uniform train load, and, in the computing department of a bridge works, it becomes necessary to take things as they are rather than as they might be.

It becomes, then, an important matter to determine the simplest manner of preparing the stress sheet, and especially within what limits it is necessary to determine the special position of the load which will produce in each member the maximum stress. With this object in view, the speaker has prepared comparative stress sheets for spans of 125, 150, 175, 200 and 300 ft., based upon loading "*E* 40" of Cooper's

* *Proceedings*, Am. Soc. C. E., March, 1899.

Mr. Cowles. Specifications, the stresses being computed by five different methods, and has tabulated the results showing the differences arising from the use of the respective methods (Table No. 5). The basis of the comparison is Method *A*, in which the position of the load is shifted, in accordance with the formulas of the author, so as to produce the maximum stress in each member. Method *B* is based upon the use of a fixed series of panel loads headed by the maximum floor-beam concentration, progressing on the bridge a panel at a time for successive shears, and covering the entire bridge for all chord moments.

The effect of the small panel load in advance of the maximum head load is disregarded. Method *C* is based upon a uniform load which will produce the same end shear as Method *A*. Method *D* is based upon a uniform load which will produce the same moment at the center as Method *A*. Method *E* is based upon a uniform load equal to the following train load with two additional concentrated loads, two panel lengths apart, which will produce the same end shear as Method *A*. The quantities given are in thousands of pounds, and represent shears not multiplied by the secant in the case of web members, and chord stresses for a depth equal to a panel length, that is, not multiplied by the tangent. A uniform panel length of 25 ft. was adopted.

The allowable limit of variance from the strictly correct stresses which may be permitted for the sake of simplifying the computation is two-fold: First, the percentage of deficiency in any member must not exceed a certain amount; second, the percentage of excess in the total weight of the structure must not exceed a certain amount, and that assumption, for an equivalent load which will result in the smallest percentages of variance and at the same time afford a simple means of preparing the stress sheet, is the one which will naturally be selected. An inspection of the table will show clearly the limit of variance involved in each of these methods, but in general it may be stated that Method *B* gives the smallest percentages, taking both considerations into account. That is, while Method *C* avoids deficient stresses in the most important members, it requires a larger excess of total material; while Method *D* results in a reduction of total weight, the deficiencies in stresses are large; and while Method *E* avoids all deficiencies in stresses, the excess of total weight is even greater than for Method *C*. The maximum deficiency in stress resulting from the use of Method *B* is, for a 150-ft. span, nothing; for a 175-ft. span, 0.1 of 1%; for a 200-ft. span, 2.1% in one member only; for a 300-ft. span, 1.8%; while the excess of total weight is, for the same spans, respectively, 0.8, 0.7, 0.4 of 1% and less than 0.1 of 1 per cent.

The permissible limit of variance is a matter which each engineer will fix for himself in accordance with his own sense of propriety, but it seems to the speaker that the percentages mentioned for Method *B*

are not excessive, when it is considered that the assumed load itself should be greater than the probable actual load by a margin at least as great as the largest of these percentages. It will be remembered also that, in the case of a new specification coming into a computer's hands, the work involved in Method *B* or its equivalent, must first be done in order to determine the uniform load which may be used in Methods *C*, *D* or *E*, and it is to be noted that still another uniform load must be determined for the floor-beam moment and the stress on the hip vertical. It may be stated here that in determining the percentage of excess weight for Methods *C* and *D* it was assumed that the stress on the hip vertical given in the table would not be used, but that this member would be brought up to proper size as just suggested. The calculation for stringers would, of course, be the same for each method. Mr. Cowles.

If the engine excess for Method *E* be determined by taking the difference between the actual engine weight and the weight of a section of the following load equal in length to the length of the engine, the resulting stresses and the excess weight will be even greater than given in the table.

In case a certain specification is likely to be used frequently, and a method of uniform load is preferred, it is naturally advantageous to prepare a series of uniform loads, corresponding to different lengths of spans, for use in computations; and it cannot be contended that this will not effect a saving in time after the series is once prepared, but the amount of this saving is, in the opinion of the speaker, much overestimated in discussing the relative merits of the different methods. An important factor affecting the simplicity of a computation is the orderly and systematic arrangement of the mathematical work, and in this matter each engineer will adopt such a system as to him will seem the quickest and easiest to obtain the desired result. The speaker has been accustomed to using Method *B*, and the system adopted by him, which is doubtless used by others, affords a reasonably quick process and one which compares favorably with the method of uniform loads. He noted recently the time occupied in making the computations for the truss members of a 300-ft. span with horizontal top chord and having 13 panels, giving odd panel lengths. The load used was "Cooper's *E* 40." Having first determined the panel length, which must be done in any case, the panel loads were determined in 17 minutes and the complete shears and moments, ready for multiplying by secant and tangent, in 18 minutes more, or 35 minutes in all.

There does not seem to be much room here for saving in time by the use of another method, even if the equivalent uniform or excess loads have already been determined.

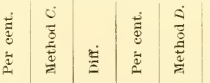
There is nothing new claimed for this method; in fact it may be

Mr. Cowles. called old-fashioned, but this comparison indicates that it gives reasonably close results and is not laborious or slow.

The speaker has not had time to make similar comparisons for subdivided or curved top trusses, but has no reason to suspect that the limits of variance would be any greater in these cases than in those investigated, and he therefore feels confident that, while railroad engineers retain a preference for a type of loading which they can compare readily with actual engines in use on their lines, satisfactory stress sheets may be prepared without undue expenditure of time and labor. He will, however, welcome such a change of attitude, on the part of these engineers, as will result in a reduction in the number of varying loadings and admit of the adoption of a limited number of standard loads, preferably uniform.

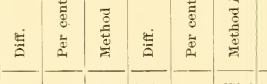
TABLE No. 5.

125-FT. SPAN.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	194.6	190.0	—4.6	2.4	199.2	+4.6	.....	194.6	.....	.....	193.7	—0.9	0.5
a c....	132.8	132.8	.....	.....	132.8	.....	.....	129.7	—3.1	2.3	132.8	.....	.....
c d....	190.0	190.0	.....	.....	199.2	+9.2	.....	194.6	+4.6	.....	193.7	+3.7	.....
B b....	75.6	75.6	.....	.....	66.4	—9.2	12.2	64.9	—10.7	14.2	77.3	+1.7	.....
C c....	36.4	40.8	+4.4	.....	39.8	+3.4	.....	38.9	+2.5	.....	40.9	+4.5	.....
B a....	132.8	132.8	.....	.....	132.8	.....	.....	129.7	—3.1	2.3	132.8	.....	.....
B c....	77.3	81.0	+3.3	.....	79.7	+2.4	.....	77.8	+0.5	.....	81.9	+4.6	.....
C d....	36.4	40.8	+4.4	.....	39.8	+3.4	.....	38.9	+2.5	.....	40.9	+4.5	.....
Excess weight..			.....	+0.5		.....	.....		.....	0.0		.....	+1.0

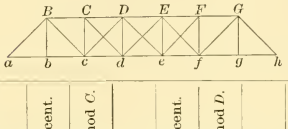
150-FT. SPAN.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	248.3	248.3	.....	.....	259.2	+10.1	.....	250.8	+2.5	.....	255.5	+7.2	.....
C D....	282.1	282.1	.....	.....	291.6	+ 9.5	.....	282.1	.....	.....	280.5	—1.6	0.6
a c....	162.0	162.0	.....	.....	162.0	.....	.....	156.7	5.3	3.3	162.0	.....	.....
c d....	248.3	248.3	.....	.....	259.2	+10.1	.....	250.8	+ 2.5	.....	255.5	+7.2	.....
B b....	75.6	75.6	.....	.....	64.8	—10.8	14.3	62.7	—12.9	17.1	77.8	+2.2	.....
C c....	62.9	67.5	+4.6	.....	64.8	+ 1.9	.....	62.7	— 0.2	0.3	68.5	+5.6	.....
D d....	29.7	34.0	+4.3	.....	32.4	+ 2.7	.....	31.4	+ 1.7	.....	34.3	+4.6	.....
B a....	162.0	162.0	.....	.....	162.0	.....	.....	156.7	— 5.3	3.3	162.0	.....	.....
B c....	107.6	110.7	+3.1	.....	108.0	+ 0.4	.....	104.5	— 3.1	2.9	111.1	+3.5	.....
C d....	62.9	67.5	+4.6	.....	64.8	+ 1.9	.....	62.7	— 0.2	0.3	68.5	+5.6	.....
D e....	29.7	34.0	+4.3	.....	32.4	+ 2.7	.....	31.4	+ 1.7	.....	34.3	+4.6	.....
Excess weight..			.....	+0.8		.....	.....		.....	—0.5		.....	+1.2

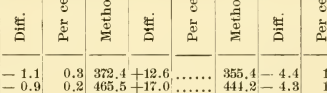
TABLE No. 5—(*Continued*).

175-FT. SPAN.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	304.8	304.3	−0.5	0.1	316.7	+11.9	.....	305.0	+ 0.2	.....	314.0	+ 9.2	.....
C D....	366.0	366.0	.....	.....	380.0	+14.0	.....	366.0	.....	.....	377.0	+11.0	.....
a c....	190.0	190.0	.....	.....	190.0	.....	.....	183.0	− 7.0	3.7	190.0	.....	.....
c d....	304.8	304.3	−0.5	0.1	316.7	+11.9	.....	305.0	+ 0.2	.....	314.0	+ 9.2	.....
d e....	363.3	366.0	+2.7	.....	380.0	+16.7	.....	366.0	+ 2.7	.....	377.0	+13.7	.....
B b....	75.6	75.6	.....	.....	63.3	−12.3	16.3	61.0	−14.6	19.3	78.0	+ 2.4	.....
C c....	90.9	94.9	+4.0	.....	90.5	− 0.4	0.4	87.1	− 3.8	4.2	95.4	+ 4.5	.....
D d....	52.6	57.9	+5.3	.....	54.3	+ 1.7	.....	52.3	− 0.3	0.6	58.8	+ 6.2	.....
B a....	190.0	190.0	.....	.....	190.0	.....	.....	183.0	− 7.0	3.7	190.0	.....	.....
B c....	136.2	138.9	+2.7	.....	135.7	− 0.5	0.4	130.7	− 5.5	4.0	139.1	+ 2.9	.....
C d....	90.9	94.9	+4.0	.....	90.5	− 0.4	0.4	87.1	− 3.8	4.2	95.4	+ 4.5	.....
D e....	52.6	57.9	+5.3	.....	54.3	+ 1.7	.....	52.3	− 0.3	0.6	58.8	+ 6.2	.....
E f....	25.0	29.1	+4.1	.....	27.2	+ 2.2	.....	26.2	+ 1.2	.....	29.4	+ 4.4	.....
Excess weight..			.....	+0.7	.....	.....	+0.9	.....	.....	.....	.....	.....	+1.4

200-FT. SPAN.

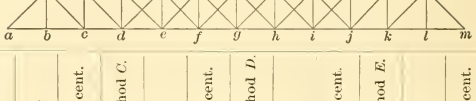


Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	359.8	358.7	− 1.1	0.3	372.4	+12.6	.....	355.4	− 4.4	1.2	370.3	+11.1	.....
C D....	448.5	447.6	− 0.9	0.2	465.5	+17.0	.....	444.2	− 4.3	1.0	459.3	+10.9	.....
D E....	473.8	463.8	−10.0	2.1	496.5	+22.7	.....	473.8	.....	.....	484.3	+10.5	.....
a c....	217.2	217.2	.....	.....	217.2	.....	.....	208.1	− 9.1	4.2	217.2	.....	.....
c d....	359.8	358.7	− 1.1	0.3	372.4	+12.6	.....	355.4	− 4.4	1.2	370.3	+11.1	.....
d e....	448.5	447.6	− 0.9	0.2	465.5	+17.0	.....	444.2	− 4.3	1.0	459.3	+10.9	.....
B b....	75.6	75.6	.....	.....	62.1	−13.7	18.1	59.2	−16.4	21.7	78.1	+ 2.5	.....
C c....	118.0	121.5	+ 3.5	.....	116.4	− 1.6	1.4	111.0	− 7.0	5.9	121.9	+ 3.9	.....
D d....	78.4	83.0	+ 4.6	.....	77.6	− 0.8	1.0	74.0	− 4.4	5.6	83.6	+ 5.2	.....
E e....	45.1	50.6	+ 5.5	.....	46.6	+ 1.5	.....	44.6	− 0.5	1.1	51.5	+ 6.4	.....
B a....	217.2	217.2	.....	.....	217.2	.....	.....	208.1	− 9.1	4.2	217.2	.....	.....
B c....	163.9	166.2	+ 2.3	.....	162.9	− 1.0	0.6	156.1	− 7.8	4.8	166.4	+ 2.5	.....
C d....	118.0	121.5	+ 3.5	.....	116.4	− 1.6	1.4	111.0	− 7.0	5.9	121.9	+ 3.9	.....
D e....	78.4	83.0	+ 4.6	.....	77.6	− 0.8	1.0	74.0	− 4.4	5.6	83.6	+ 5.2	.....
E f....	45.1	50.6	+ 5.5	.....	46.6	+ 1.5	.....	44.6	− 0.5	1.1	51.5	+ 6.4	.....
F g....	21.5	25.5	+ 4.0	.....	23.3	+ 1.8	.....	22.3	+ 0.8	3.7	25.8	+ 4.3	.....
Excess weight..			.....	+ 0.4	.....	.....	+ 1.0	.....	.....	.....	.....	.....	+1.4

Mr. Cowles.

TABLE No. 5—(*Concluded*).

300 FT. SPAN.



Member.	Method A.	Method B.	Diff.	Per cent.	Method C.	Diff.	Per cent.	Method D.	Diff.	Per cent.	Method E.	Diff.	Per cent.
B C....	571.2	569.3	— 1.9	0.3	586.4	+15.2	.....	542.3	—28.9	5.1	585.5	+14.3	.....
C D....	765.1	763.5	— 1.6	0.2	791.6	+26.5	.....	732.1	—33.0	4.3	789.0	+23.9	.....
D E....	899.2	885.0	—14.2	1.6	938.2	+39.0	.....	867.7	—31.5	3.5	933.0	+33.8	.....
E F....	965.3	948.2	—17.1	1.8	1026.2	+60.9	.....	949.0	—16.3	1.7	1017.5	+52.2	.....
F G....	976.2	962.9	—13.3	1.4	1055.5	+79.3	.....	976.2	.....	.....	1042.5	+66.3	.....
a c....	322.5	322.5	.....	.....	322.5	.....	.....	298.3	—24.2	7.5	322.5	.....	.....
c d....	571.2	569.3	— 1.9	0.3	586.4	+15.2	.....	542.3	—28.9	5.1	585.5	+14.3	.....
d e....	765.1	763.5	— 1.6	0.2	791.6	+26.5	.....	732.1	—33.0	4.3	789.0	+23.9	.....
e f....	899.2	885.0	—14.2	1.6	938.2	+39.0	.....	867.7	—31.5	3.5	933.0	+33.8	.....
f g....	965.3	948.2	—17.1	1.8	1026.2	+60.9	.....	949.0	—16.3	1.7	1017.5	+52.2	.....
B b....	75.6	75.6	.....	.....	58.6	—17.0	22.5	54.2	—21.4	28.3	78.5	+ 2.9	.....
C c....	223.0	225.3	+ 2.3	.....	220.0	— 3.0	1.4	203.4	—19.6	8.8	225.5	+ 2.5	.....
D d....	179.9	183.0	+ 3.1	.....	175.9	— 4.0	2.2	162.7	—17.2	9.6	183.3	+ 3.4	.....
E e....	140.9	144.8	+ 3.9	.....	136.8	— 4.1	2.9	126.6	—14.3	10.1	145.2	+ 4.3	.....
F f....	106.2	110.8	+ 4.6	.....	102.6	— 3.6	3.4	94.9	—11.3	10.6	111.3	+ 5.1	.....
G g....	76.0	81.0	+ 5.0	.....	73.3	— 2.7	3.6	67.8	— 8.2	10.8	81.5	+ 5.5	.....
B a....	322.5	322.5	.....	.....	322.5	.....	.....	298.3	—24.2	7.5	322.5	.....	.....
B c....	270.2	271.8	+ 1.6	.....	268.7	— 1.5	0.6	248.6	—21.6	8.0	271.9	+ 1.7	.....
C d....	223.0	225.3	+ 2.3	.....	220.0	— 3.0	1.4	203.4	—19.6	8.8	225.5	+ 2.5	.....
D e....	179.9	183.0	+ 3.1	.....	175.9	— 4.0	2.2	162.7	—17.2	9.6	183.3	+ 3.4	.....
E f....	140.9	144.8	+ 3.9	.....	136.8	— 4.1	2.9	126.6	—14.3	10.1	145.2	+ 4.3	.....
F g....	106.2	110.8	+ 4.6	.....	102.6	— 3.6	3.4	94.9	—11.3	10.6	111.3	+ 5.1	.....
G h....	76.0	81.0	+ 5.0	.....	73.3	— 2.7	3.6	67.8	— 8.2	10.8	81.5	+ 5.5	.....
H i....	50.4	55.3	+ 4.9	.....	48.9	— 1.5	3.0	45.2	— 5.2	10.3	55.9	+ 5.5	.....
I j....	29.0	33.7	+ 4.7	.....	29.3	+ 0.3	.....	27.1	— 1.9	6.6	34.5	+ 5.5	.....
Excess weight..	.....	.....	.....	0.0	.....	.....	+ 1.2	.....	.....	.....	.....	.....	+1.7

Mr. Scofield.

E. M. SCOFIELD, Assoc. M. Am. Soc. C. E.—For several years the speaker's time has been devoted to the designing of truss bridges. He has used both methods of calculation, and is not in favor of the method of wheel loads. For each new engine loading it is necessary to make a moment table, which has to be checked carefully, and, up to this point, the two methods are the same. Aside from that, the speaker can compute uniform loads from end shear and figure the stresses in a bridge in about one-quarter to one-tenth of the time required by the exact wheel-load method, thus saving much time and largely avoiding danger from mistakes.

By the uniform method, after checking the end shear, all the stresses in the bridge can be found in from 15 to 20 minutes, and checked beyond all possibility of error. By the wheel-load method the speaker has never been able to make the calculations and check them in such short time; it generally takes from four to ten times as long, and then does not afford such a ready means of checking. In fact, the speaker has seldom made such calculations by wheel loads without making in parallel columns similar calculations by uniform load as a check; and he finds that such a method of checking is more reliable than to attempt to check a wheel-load calculation.

In using the wheel-load method of calculation, the number of manipulations is so great that a man who is figuring with various engines and various specifications is in great danger of making mistakes; whereas the simplicity of the equivalent uniform-load method is such that the calculations can be checked in a very short time. Mr. Scofield.

While there are many different methods of determining equivalent uniform loads, that which bases such loads on the shear in the end panel, is, in the opinion of the speaker, most satisfactory for truss calculations.

All engineers do not agree with this view, but a uniform load based on the shear in the end panel will give stresses which are exact for the end post and nearly exact for the first tie, the error being always on the side of safety and increasing slowly toward the center, and being greatest in the counters where the tonnage involved is small and where an excess of metal is to be commended. This method also gives in the chords a little too much sectional area, but, judging from actual tests of bottom chords, under everyday conditions, a little excess of metal in those members is likely to be well placed.

The argument for exact wheel-loads is altogether too refined, because the engine specified may never pass over the bridge, and the variation due to the use of a uniform load will not exceed 4% in ordinary cases and will not be nearly so much in most of the members where the weight is concentrated.

In the opinion of the speaker, it is well that wheel loads be specified simply because they are more intelligible, especially to the buyer. A railroad engineer who is not dealing with bridges all the time, or the engineer whose time is mostly devoted to other kinds of work, will prefer to have wheel loads specified because they mean something definite to him, which the uniform load does not.

The speaker believes that, in addition to specifying a typical engine or an actual engine, with its following load, a uniform load obtained in a certain, definite way should also be specified and required to be used in making the calculations. The speaker would specify loads based on the center moment for stringers and floor beams and on the moment at the quarter point for girders, and on the shear in the end panel for trusses. This method would give a better bridge and would save time in making the calculations, which would, in the end, be a saving to the buyer. There are other methods for obtaining equivalent loads which are not without merit and are alike in being preferable to the wheel-load method. It would be an additional advantage if the equivalent loads for spans varying by small intervals were given.

HENRY B. SEAMAN, M. Am. Soc. C. E.—Any movement toward the adoption of a train load with a single concentration, instead of what is termed a "typical" engine, but which has really become an equivalent typical engine, should be encouraged. About fifteen years ago it Mr. Seaman.

Mr. Seaman. was the custom to specify all the various types of engines which might in any way produce the maximum strain under varying conditions. Calling to mind especially the Pennsylvania Railroad Specifications, which were presented to the Society about that time by Joseph M. Wilson, M. Am. Soc. C. E., three typical engines were used. These were truly typical, and represented very closely the actual weights and wheel spacing of the various types of engines in use, such as the consolidation, the passenger and the switch engine. But as general specifications were written for adoption upon various roads, it was found expedient to use one engine diagram which would produce strains approximately equivalent to the maximum from the three types. A consolidation engine diagram was outlined for this purpose, and this might properly be termed an equivalent typical engine, *i. e.*, producing strains approximately equivalent to the three types formerly used.

Some ten years ago the speaker had occasion to investigate the weights of engines for C. W. Buchholz, M. Am. Soc. C. E., Chief Engineer of the Erie Railroad, and the result of this study was given in a discussion of the paper on "Some Disputed Points in Railway Bridge Designing.*" There was no engine then in use with a weight on the pony wheel exceeding 16 000 lbs.; and the weights on an axle of the tender in no case would exceed 23 000 lbs., with full weight of water, and with sides boarded up to carry an extra heavy load of coal. To-day the typical engines, or rather the equivalent typical engines, are shown with 20 000 lbs. on the pony wheel, and in some instances with 26 000 lbs. on each tender axle, which is a variation of 10 to 25% from actual conditions. In other words, we are designing bridges for loads which do not exist, and which cannot come upon them.

Mr. Pegram's discussions of the papers "What is the Life of an Iron Railroad Bridge?" and "American Railroad Bridges," as well as reviews in some of the text books, indicate that a uniform train load with a single concentration may produce strains generally within 10% of the engine specified. This would seem about as close as the equivalent typical engine, generally used, would come to the actual load, and, if so, much labor would be saved by the adoption of the single concentration method. The question, therefore, would appear to be, not whether the method of wheel concentration should be continued, but what equivalent load should be used? It should give results approximately equivalent to those obtained in practice; but at the same time the method should be so concise as to permit of its insertion in a specification.

It is, perhaps, unnecessary to discuss further the fatigue formula. The Launhardt formula, which is now used by a number of the larger trunk lines, is a precise interpretation of precise experiments which are never precisely repeated in practice. A study of these same

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 77.

experiments indicates that if we consider the destructive effect of live strain as about twice that of dead strain, we will obtain even better general results than by the Launhardt formula, and since this general ratio of 2 to 1 has long been adopted in the better bridge practice, there is no justification for the use of the Launhardt formula. Mr. Seaman.

J. P. Snow, M. Am. Soc. C. E. (by letter).—The assumptions which it is necessary to make on the matters of typical loads, impact, proper unit strains, fatigue formulas, structural motion, personal predelictions as to column formulas, assessment of riveted members, proper differences for soft and medium steel and many other elements of bridge design which depend on the personal views of the designer, render any hair-splitting nicety absurd, either in computing the strain-sheet or in applying the various rules for deducing sections after the strain-sheet is determined. Mr. Snow.

The railroad official who buys bridges and who does not know how to design them must have a method which he can trust for specifying what he wants, both as to loading and the translation of that load into the final sections of the bridge members.

Without question, a definite engine loading is understood much more readily by such an official than a table or diagram of uniform loads; and Cooper's specification or some other generally accepted one is confidently referred to by him with the supposition that it gives a clear-cut and sufficient set of rules to guide all competing bridge builders in translating the prescribed load into the desired bridge.

The usefulness of the engine loading disappears if the bridge buyer designs his own bridges. The writer has used the uniform-load method exclusively for the last 15 years, both for designing new bridges and examining old ones, and finds it in every way satisfactory and ample, in all cases, to cover the whole ground.

The competing bridge builders who must prepare designs and tenders for the various loadings and specifications offered have the worst of it.

They must find the absolute maximum strains, or the consulting engineer to whom the tenders are referred will throw out their plans; and they must not make such liberal assumptions as will cause them to use more metal than is absolutely needed, else their keener competitors will underbid them. It is a case of full and exact figuring to the last possible permutation with them on every specification offered, and all this with the dreary knowledge that not more than one bid in a dozen will result in an actual contract.

Unless the bridge buyers can be brought to specify uniform loads for their bridges it is difficult to see how the overworked builders can be helped. One set of loads for moments and another for shears, as some have recommended, can hardly appeal to their sense of consistency. A uniform load plus a single rolling load, as many ad-

Mr. Snow. vocate, will not afford them much chance for comparison with familiar structures or train weights.

A single set of uniform loads giving the same center moment as the maximum wheel loads assumed serves as the basis of all the writer's computations and seems to him to be the simplest and most easily comprehended of any that he has seen advocated. The floor-beam load is, of course, the panel length multiplied by the load per foot corresponding to a span equal to twice the panel length. This load is used as a head load in getting the web shears, the following panel loads being those used for the chord moments. By neglecting the partial load ahead of the point under consideration an excessive shear is obtained, but many good designers advocate this practice, and, in the writer's opinion, it is justified, from the fact that under a fast moving train the web members appear to be much more disturbed than the chords.

Shear on longitudinal girders is obtained by using 57% of the total live load for the end reaction and 19% for the center shear. Intermediate shears are proportional to these. The actual maximum shear produced by a set of wheel loads expressed graphically is a flat curve; but the straight line given by the above percentages covers its maximum points and gives a small surplus at the quarter point of the span.

A system of loads for all spans cannot be properly made up from a single class of engines. Heavy passenger engines with two driving axles, or tank shifters on two axles close together, must be used for spans of less than 20 ft., while consolidation engines will govern on long spans. Car loads are so heavy at present that the customary legend of two engines followed by a certain train load may well be abandoned, and a continuous string of typical engines be used in obtaining moments.

It appears to the writer that if prominent bridge engineers could bring themselves to re-write their specifications, giving simple tables or curves of the uniform loads per foot corresponding to various sets of typical wheel concentrations, and give rules for using these loads for all cases, that the bridge buyers would quickly fall into line and the situation would be relieved.

The speaker would vote unreservedly in favor of discontinuing the method of wheel concentrations in determining the stresses in railroad bridges.

Mr Thacher. EDWIN THACHER, M. Am. Soc. C. E. (by letter).—The first specification the writer ever saw giving wheel concentrations as a basis for the calculation of stresses due to live load was issued by the Cincinnati Southern Railroad about 25 years ago. Previous to that time, uniform loads had invariably been used in calculations; unit stresses being varied arbitrarily to suit length of span and location of member.

Mr. Thacher.

This specification gave a live-load diagram of two 10-wheel engines, 36 tons in 12 ft. and 66 tons in 50 ft., followed by 1 820 lbs. per lineal foot. The writer was then employed as Assistant Engineer for the Louisville Bridge and Iron Company, of Louisville, Ky., of which the late Albert Fink, Past-President, Am. Soc. C. E., was President. The specification was received by the morning's mail, together with a letter inviting proposals for several bridges at an early date.

The combined talent of the office was assembled, and that specification was thoroughly discussed. It was voted unanimously that the specification was absurd; that an engine of such weight had never been heard of, and would never be used on the Cincinnati Southern Railroad or any other; that to design bridges for such a load was a reckless waste of money, and that to specify such a load was an outrage. After this feeling had had time to cool off, it was fully realized that this load must be used in calculations, and that the estimates and strain sheets must be prepared speedily, and the all-important question was how to do it. This was before the days of moment tables, or moment diagrams, and before any investigations had appeared giving the exact location of load for the maximum stress in any member.

Late in the afternoon of that day, R. Montfort, M. Am. Soc. C. E., now Chief Engineer of the Louisville and Nashville Railroad, suggested a method of calculation which was at once adopted, and which gave results near enough for all practical purposes. The first driver was placed over a panel point and the panel weights resulting from such location were used in the calculation, and, for double intersection bridges, in which the stresses are somewhat ambiguous at best, not much improvement can be made on that method to-day.

Specifications giving wheel concentrations, once introduced, came rapidly to the front, and in a short time all the railroads were in line, each with a different kind of load, and the bridge companies shouldered their burden, which they have continued to carry ever since.

The writer has been satisfied for many years past that the method of wheel concentrations in the calculation of bridge stresses was a waste of time, and has done what he could to correct the evil. In the specifications of the Keystone Bridge Company of 1887 and 1890 are given tables for equivalent uniform loads for engines of ten different weights, from 44-ton narrow gauge, to 112-ton decapod, and for spans from 10 ft. to 300 ft. in length. For spans of 70 ft. and less (70 ft. being about the maximum length of plate girders used at that time) the uniform loads are such as will give a maximum center moment; and for spans of 75 ft. and upward, such as will give a maximum end shear. These tables have been quite extensively used, and give results practically the same as the wheel loads, and in one-tenth the time. To prepare a moment table for a new loading is some labor, but when

Mr. Thacher. once prepared, the equivalent uniform load for any span can be found, by aid of the slide rule, in a few minutes.

It was the practice of the writer for some years to calculate bridges, using wheel concentrations, and check by the method of uniform loads, but gradually he came to use uniform loads altogether, except in special cases, generally confined to cases in litigation or reconstruction.

It appears to the writer that the railroad companies would, on proper representation, agree on four or five typical loads, ranging from light to the heaviest in use, some one of which they would be willing to adopt for their bridges. By an expert bridge engineer these could soon be tabulated for convenient use, giving the moments and shears required in girder calculations, and the equivalent loads to be used in the calculation of trusses, thus saving an immense amount of useless labor repeated over and over again. The writer is satisfied that this result will never be accomplished by the efforts of any one engineer, however meritorious his scheme, but believes it can be accomplished by a Special Committee of this Society.

Few engineers believe that Launhardt's formula should be used in designing bridges on account of any fatigue of the material due to repeated application of load, but the writer believes that a modification of this formula is the best thing yet devised for taking account of impact in the distribution of bridge material.

The writer prefers the form, $a\left(1+\frac{\text{Min.}}{\text{Max.}}\right)$. Cooper's specification reduces to $b\div\left(1+\frac{\text{L. L.}}{\text{Max.}}\right)$; and both give practically the same result. It is generally conceded that live load has a worse effect on a bridge than dead load, and the above formulas assume that it has double the effect, but they can be varied to suit any ratio desired, and the results are consistent with each other. The writer could never see any merit in the arbitrary rule-of-thumb methods which add 25% for girders 25 ft. long, and 15% for girders 25 ft. 1 in. long; 50% to this and 10% to that; and he will continue to advocate the above modification of Launhardt's formula until something better is proposed.

Mr. Waddell. J. A. L. Waddell, M. Am. Soc. C. E. (by letter).—In 1891 the writer presented to this Society a paper entitled "Some Disputed Points in Railway Bridge Designing,"* in which he advocated the adoption of a few standard train loads for railroad bridges, instead of the numerous ones then in use, offering instead a set of live loads for discussion, and urging that the "Equivalent Uniform-Load Method" be adopted instead of the tedious, burdensome, hair-splitting and heart-breaking method of "Wheel Concentrations."

The thorough discussion which this paper received, and the favor-

* *Transactions*, Am. Soc. C. E., Vol. xxvi, p. 77.

able opinions of most of those who discussed the questions, and of the technical press, induced the writer to carry farther his investigations and to make an earnest endeavor to establish a few standard live loads and the simpler method of computing stresses. Mr. Waddell.

The reviews in the technical press started a series of papers by the writer and others, which were printed in the *Railroad Gazette* and *The Engineering Record*.

These showed conclusively that the "Equivalent Uniform-Load Method" gives results which are accurate enough for all practical purposes, and which are more accurate than those which are obtained by either the "Single-Concentration Method" or the "Double-Concentration Method." Concerning the accuracy of this, there appears to be still some doubt in the minds of a few members of the Society, as shown by opinions expressed in this discussion, so the writer gives herewith certain tabulated results of computations made some years ago for the special purpose of comparing the three alternative methods with the so-called exact method, all tabulated figures having been checked by two computers working entirely independently of each other. The span assumed for comparison was one of 200 ft., which is not far from the average span length for truss bridges in America; and the live loads adopted are those of Classes *Z* and *U* of the writer's "Compromise Standard System of Live Loads for Railroad Bridges."

For trusses, the equivalent uniform load used in each case is an average of those figured for each panel point. This, for nearly all spans, is almost exactly the same as the equivalent for the quarter points.

TABLE No. 6.—Percentages of Error in Moments at Centers of Plate-Girder Spans, Found by Equivalent-Uniform, Single-Excess and Double-Excess Systems.

Span length, Feet.	Class *Z*.			Class *U*.		
	Equivalent uniform system.	Single-excess system.	Double-excess system.	Equivalent uniform system.	Single-excess system.	Double-excess system.
15....	none.	14.0 safety.	14.0 safety.	none.	19.9 safety.	19.9 safety.
20....	"	7.8 "	7.8 "	"	11.8 "	11.8 "
25....	"	4.2 "	4.2 "	"	5.3 "	5.3 "
30....	"	2.1 "	72.1 "	"	1.2 "	1.2 "
35....	"	0.7 "	0.7 "	"	1.3 danger.	1.3 danger.
40....	"	0.5 "	0.5 "	"	2.3 "	2.3 "
45....	"	0.3 "	0.3 "	"	2.8 "	2.8 "
50....	"	0.0 "	0.0 "	"	3.2 "	3.2 "
60....	"	0.4 danger.	0.4 danger.	"	2.7 "	2.7 "
70....	"	0.1 safety.	0.1 safety.	"	2.2 "	2.2 "
80....	"	0.0 "	0.0 "	"	1.9 "	1.9 "
90....	"	0.1 danger.	0.1 danger.	"	1.7 "	1.7 "
100...	"	0.9 "	0.9 "	"	1.2 "	1.2 "

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TABLE No. 7—(*Continued*).

300		Batter brace	0.4 danger.	1.4 danger.	0.3 danger.	0.7 safety.	1.5 danger.	1.3 danger.
		1st main diag	0.6 "	0.9 "	0.2 safety.	0.1 "	0.8 "	0.8 "
		2d "	0.6 "	0.0 "	1.1 "	0.7 danger.	0.3 "	0.5 "
		3d "	0.9 "	0.8 safety.	1.8 "	1.5 "	0.6 safety.	0.1 safety.
		4th "	1.0 "	2.0 "	2.9 "	2.3 "	1.9 "	1.0 "
		5th "	0.7 "	4.1 "	4.7 "	2.8 "	4.1 "	2.5 "
		1st counter	0.4 safety.	7.5 "	7.5 "	3.0 "	7.4 "	4.7 "
		2d "	1.9 "	12.5 "	11.2 "	2.5 "	12.9 "	8.0 "
	1st top chord		0.1 danger.	1.1 danger.	0.1 danger.	0.7 safety.	1.4 danger.	1.4 danger.
	2d "		0.5 safety.	0.5 "	0.4 safety.	1.0 "	1.2 "	1.4 "
	3d "		0.7 "	0.3 "	0.4 "	1.6 "	0.6 "	0.9 "
	4th "		0.7 "	0.3 "	0.3 "	1.8 "	0.4 "	1.0 "
	5th "		0.6 "	0.4 "	0.1 danger.	1.8 "	0.3 "	1.3 "
400		Batter brace	0.1 danger.	1.3 "	0.3 "	0.7 "	1.5 "	1.1 "
		1st main diag	0.2 "	1.1 "	0.0 safety.	0.4 "	1.2 "	0.8 "
		2d "	0.4 "	0.7 "	0.4 "	0.2 danger.	1.0 "	0.7 "
		3d "	0.5 "	0.3 "	0.8 "	0.8 "	0.7 "	0.5 "
		4th "	0.8 "	0.1 safety.	1.2 "	1.4 "	0.3 "	0.2 "
		5th "	0.9 "	0.6 "	1.8 "	2.0 "	0.3 safety.	0.3 safety.
		6th "	1.1 "	1.4 "	2.5 "	2.6 "	1.1 "	0.9 "
		7th "	1.0 "	2.5 "	3.5 "	3.2 "	2.3 "	1.8 "
		1st counter	0.8 "	4.1 "	5.0 "	3.9 "	3.6 "	2.7 "
		2d "	0.7 "	6.0 "	6.6 "	4.7 "	5.6 "	4.0 "
		3d "	1.3 safety.	10.5 "	10.5 "	4.9 danger.	9.0 "	6.1 "
	1st top chord		0.1 "	1.1 danger.	0.1 danger.	0.7 safety.	1.5 danger.	1.2 danger.
	2d "		0.3 "	0.9 "	0.1 safety.	0.8 "	1.4 "	1.1 "
	3d "		0.6 "	0.7 "	0.2 "	0.9 "	1.3 "	1.1 "
	4th "		0.6 "	0.6 "	0.2 "	1.3 "	1.2 "	1.1 "
	5th "		0.6 "	0.6 "	0.2 "	1.4 "	0.9 "	0.9 "
	6th "		0.6 "	0.6 "	0.1 "	1.4 "	0.8 "	0.9 "
	7th "		0.6 "	0.6 "	0.1 danger.	1.4 "	0.8 "	1.1 "
	Bottom chord		0.1 danger.	1.3 "	0.3 "	0.7 "	1.5 "	1.1 "

TABLE No. 7.—Percentages of Error in Stresses, found by Equivalent-Uniform, Single-Excess and Double-Excess Methods of Loading.

Span length.	Chord members.	Web members.	Class Z. Equivalent-uniform system.	Class Z. Single-excess system.	Class Z. Double-excess system.	Class U. Equivalent-uniform system.	Class U. Single-excess system.	Class U. Double-excess system.
100 ft.		Batter brace	0.9 danger.	4.6 safety.	0.9 safety.	1.8 danger.	7.5 safety.	1.2 danger.
		1st main diag.	6.1 safety.	20.2 "	8.5 "	4.5 safety.	26.0 "	5.3 safety.
		1st counter	19.9 "	60.0 "	39.0 "	13.5 "	66.0 "	39.0 "
	1st top chord		3.7 "	9.6 "	0.9 "	4.9 "	15.0 "	0.8 danger.
	Bottom chord		0.9 danger.	4.6 "	0.9 "	1.8 danger.	7.5 "	1.2 "
150 ft.		Batter brace	1.6 "	0.1 danger.	0.1 danger.	1.1 "	0.9 "	1.7 "
		1st main diag.	0.3 "	4.3 safety.	3.1 safety.	0.6 "	6.0 "	1.3 safety.
		2d "	3.2 safety.	12.6 "	9.0 "	1.4 safety.	15.3 "	6.0 "
		1st counter	10.2 "	30.1 "	17.0 "	7.2 "	35.0 "	12.8 "
	1st top chord		1.1 "	2.8 "	1.7 "	0.6 "	2.5 "	1.4 danger.
	2d "		1.3 "	2.6 "	0.3 "	2.6 "	4.7 "	1.6 "
	Bottom chord		1.6 danger.	0.1 danger.	0.1 danger.	1.1 danger.	0.9 "	1.7 "
200 ft.		Batter brace	1.1 "	1.1 "	0.3 "	0.2 "	0.8 danger.	1.7 "
		1st main diag.	1.0 "	0.7 safety.	1.2 safety.	0.7 "	1.3 safety.	0.3 "
		2d "	0.5 "	3.4 "	3.4 "	2.8 "	2.6 "	0.1 "
		3d "	0.8 safety.	7.4 "	6.7 "	1.3 "	8.9 "	4.1 safety.
		1st counter	5.9 "	18.7 "	14.6 "	2.6 safety.	20.7 "	11.0 "
		2d "	11.8 "	35.5 "	22.2 "	7.5 "	40.3 "	17.0 "
	1st top chord		0.4 "	0.4 "	0.9 "	0.7 "	0.1 "	1.3 danger.
	2d "		0.7 "	0.7 "	0.7 "	1.6 "	1.1 "	1.0 "
	3d "		0.6 "	0.6 "	0.2 danger.	2.2 "	1.7 "	1.4 "
250 ft.		Batter brace	0.8 danger.	1.4 danger.	0.3 "	0.4 "	1.3 danger.	1.5 "
		1st main diag.	0.9 "	0.4 "	0.5 safety.	0.3 danger.	0.3 "	0.7 "
		2d "	0.9 "	0.9 safety.	1.7 "	1.3 "	0.8 safety.	0.1 "
		3d "	0.9 "	2.5 "	3.1 "	2.1 "	2.7 "	1.1 safety.
		4th "	0.3 "	5.5 "	5.5 "	2.4 "	5.9 "	3.1 "
		1st counter	1.9 safety.	11.1 "	9.9 "	1.4 "	11.8 "	6.8 "
		2d "	5.9 "	20.9 "	17.0 "	1.7 safety.	22.9 "	13.0 "
	1st top chord		0.4 "	0.2 danger.	0.8 "	0.6 "	1.1 danger.	1.5 danger.
	2d "		0.5 "	0.1 "	0.6 "	1.1 "	0.6 "	1.3 "
	3d "		0.7 "	0.1 safety.	0.5 "	1.6 "	0.0 "	1.2 "
	4th "		0.5 "	0.1 danger.	0.1 danger.	1.9 "	0.3 safety.	1.5 "

Mr. Waddell. TABLE No. 8.—PERCENTAGES OF ERROR IN END SHEARS, FOUND BY EQUIVALENT-UNIFORM, SINGLE-EXCESS, AND DOUBLE-EXCESS SYSTEMS.

Span length. Feet.	CLASS *Z*.			CLASS *U*.		
	Equivalent uniform system.	Single-excess system.	Double-excess system.	Equivalent uniform system.	Single-excess system.	Double-excess system.
15....	none.	1.9 safety.	22.5 danger.	none.	45.4 safety.	3.0 danger.
20....	"	6.0 "	16.7 "	"	33.3 "	10.0 "
25....	"	7.6 "	13.2 "	"	22.8 "	12.3 "
30....	"	5.8 "	12.8 "	"	18.1 "	13.3 "
35....	"	2.8 "	13.9 "	"	14.3 "	14.3 "
40....	"	1.7 "	13.8 "	"	12.0 "	14.4 "
45....	"	0.7 "	13.5 "	"	10.5 "	14.1 "
50....	"	0.6 "	12.5 "	"	9.45 "	13.5 "
60....	"	0.5 "	7.8 "	"	7.9 "	8.4 "
70....	"	1.5 danger.	6.8 "	"	4.0 "	7.6 "
80....	"	3.2 "	6.6 "	"	0.7 "	7.9 "
90....	"	4.3 "	6.4 "	"	1.5 danger.	8.0 "
100...	"	5.1 "	6.4 "	"	2.7 "	7.9 "

In these tables the best possible showing was made for both the "Single-Concentration Method," and the "Double-Concentration Method," as an inspection will show.

The single concentrated load for each class was found by ascertaining, for each main diagonal and each chord section of a 150-ft., a 300-ft., and a 500-ft. span, the single concentrated excess load, which, in addition to the uniform car-load, would produce the same shear or moment as that found by the wheel-concentration method. These excesses for each span were then averaged, and, finally, the average of these averages was taken as the most nearly correct excess load for main-truss members. The results were, respectively, for Classes *Z* and *U*, 39 372 lbs. and 88 368 lbs.

Next, the average single concentrations for all plate-girder spans between 20 ft. and 100 ft. for both classes were determined, and found to be, respectively, 24 577 lbs. and 51 750 lbs. Now, as the floor system is quite as important as the trusses, it was deemed proper to average these concentrations with those found for main-truss members, in order to determine the final single concentration for each class, thus making the said final concentrations, in round numbers, 32 000 lbs. for Class *Z* and 70 000 lbs. for Class *U*.

On applying these to both floor systems and trusses, the errors found were so great as to show conclusively that no single concentration can be assumed which, combined with the constant car load, will give results that even approximate to those found by the concentrated wheel-load method.

It became necessary, therefore, to adopt one concentration for the floor system and another for the trusses, in order to make any show-

ing at all for the system. For Class *Z* there were used 25 000 lbs. for the floor system and 40 000 lbs. for the trusses, and for Class *U* 50 000 lbs. and 90 000 lbs., respectively. On applying these, it was found necessary to make still another compromise, viz., to use the truss concentration in computing end shears in plate-girder spans and in floor systems. Mr. Waddell.

Adopting these compromises, and making calculations thereby, gave the percentages of error shown in Tables Nos. 6, 7 and 8, in the columns headed "Single-Excess system."

The double concentrated loads for the two classes were found by making some extensive calculations. These loads varied from 25 000 lbs. for Class *Z* to 50 000 lbs. for Class *U*. By using these, the percentages of error in Tables Nos. 6, 7 and 8, under the headings "Double-Excess System," were ascertained.

A study of Table No. 7 shows that the equivalent uniform-load method is much more accurate for main-truss members than either the single or the double-concentration method, and, referring to Table No. 6, it is seen that, while the equivalent-uniform load gives exact results (or as nearly so as a due consideration for regularity in the curves will permit), both of the other methods, especially for short spans and stringers of ordinary length, give large errors, notwithstanding the fact that there were adopted different concentrations for floor systems and trusses.

In order to eliminate entirely the personal equation from these tables, Table No. 9 was prepared by collecting the greatest summations of percentages of error for both chords and webs, counters being ignored:

TABLE No. 9.—Variations in Percentages of Error, as given by Equivalent-Uniform, Single-Excess and Double-Excess Systems.

Span Length, Feet.	Members.	Class *Z*.			Class *U*.		
		Equivalent uniform system.	Single-excess system.	Double-excess system.	Equivalent uniform system.	Single-excess system.	Double-excess system.
100..	Chord members.	4.6	5.0	.....	6.7	7.5	0.4
	Web "	7.0	15.6	7.6	6.3	18.5	6.5
150..	Chord "	0.2	0.2	1.4	2.0	2.2	0.2
	Web "	4.8	12.7	9.1	2.5	14.4	7.7
200..	Chord "	0.3	0.3	1.1	1.5	1.6	0.4
	Web "	1.9	8.9	7.0	2.6	9.7	5.8
250..	Chord "	0.3	0.3	0.9	1.3	1.4	0.3
	Web "	0.6	6.9	5.8	2.8	7.2	4.6
300..	Chord "	0.8	0.8	0.5	1.1	1.1	0.5
	Web "	0.6	5.5	5.0	3.5	5.6	3.8
400..	Chord "	0.5	0.5	0.3	0.7	0.7	0.3
	Web "	1.0	3.8	3.8	3.9	3.9	2.9

When the extreme errors were of opposite kinds, their arithmetical sum was taken, but when they were of the same kind their numerical difference was taken.

A study of Table No. 9 shows that, as far as chord stresses are con-

Mr. Waddell. cerned, there is but little to choose from between the three methods, but that, in respect to web members, the equivalent uniform-load method is very much more accurate than either the single-excess or the double-excess method.

Summarizing, it is evident that the following have been proved:

1st. The equivalent uniform-load method, for all bridges of types sanctioned by the best modern practice, gives stresses agreeing sufficiently well for all practical purposes with those found by the concentrated wheel-load method.

2d. That there is no combination of a constant car load per lineal foot and a single concentrated load which will give, for all members of trusses and floor systems, stresses which will agree even approximately with those found by the concentrated wheel-load method.

3d. That even if for any system of loading two separate single concentrations be adopted, one for floor systems and the other for main-truss members, the said single concentrations cannot be adjusted so as to give results agreeing sufficiently well for practical purposes with those found by the concentrated wheel-load method.

4th. That if two engine excesses placed about 50 ft. apart at panel points be adopted in connection with a constant car load per lineal foot, the stresses obtained thereby in members of floor systems and trusses do not agree sufficiently well, for practical purposes, with those obtained by the concentrated wheel-load method.

Now, on the strength of this showing, what is the use of talking any more about using a constant car load per lineal foot headed by either one or two concentrations? Either of the last two methods requires, in making calculations of stresses, fully double the time and labor required by the equivalent uniform-load method, and is by no means as accurate.

There is no method of computing live-load stresses so simple and easy as that which involves the use of properly figured uniform loads, so, why not adopt it for good and all, if we are to abandon in future the use of the "Wheel-Concentration Method"?

And surely the time has come to abandon the latter method forever, if one may judge by the unanimous opinion of those who took part in the oral discussion of the question.

In respect to Mr. Morison's proposed method, it must be evident to all that instead of simplifying matters it would complicate them worse than ever.

The objections which railroad engineers offer to using live loads which they cannot understand readily, and which, at least in their minds, do not approximate actual train loads, is well founded.

Let each railroad engineer who desires to have a bridge designed choose a combination of two locomotives and a train of cars of indefinite length weighing a certain number of pounds per lineal foot, and, on the same diagram upon which this loading is drawn to scale, let there be

drawn also the curves for equivalent loads and for end shears in plate-girder spans; and let there be written thereon a note to the effect that the curves are to be used instead of the actual wheel concentrations. Mr. Waddell.

But, instead of each engineer having a live load of his own, why not provide a variety of live loads differing from each other by uniform gradations from which anyone can choose a load to suit his requirements?

"In November, 1892, the author sent a circular-letter to all the chief engineers of railroads in the United States and Canada who were members (in any grade) of the American Society of Civil Engineers, and to every other member of that society connected with or specially interested in the designing, building or operating of railroad bridges. This letter solicited a ballot on certain 'Disputed Points in Railway Bridge Designing,' foremost among which were those of standard live loads and a simple equivalent method for computation. The number of responses received was as great as could have been expected; and the result was that about 82% of those who voted favored, and 18% opposed, the adoption of 'A Standard System of Live Loads for Railway Bridges' similar to that proposed by the author. Eighty-two per cent. also of those who voted were in favor of abandoning the 'Concentrated Wheel-Load Method,' and 18% were in favor of retaining it. Of the former, 78% favored the 'Equivalent-Uniform Load Method,' and 22% were in favor of either the 'Single' or the 'Double Concentration Method.' A number of gentlemen who responded made valuable suggestions in respect to the standard system of live loads propounded, and by the aid of these the author prepared a proposed 'Compromise Standard System of Live Loads for Railway Bridges,' and submitted the same, as before, for final ballot in May, 1893.

"The number of replies received showed that great interest was taken in the question; and the result of the ballot was 90% in favor and 10% opposed to the proposed standard.

"Next the pamphlet was published and distributed quite generally among those engineers interested in the subject of bridges, a copy being sent not only to every one who had replied to the ballots, but also to every railroad chief engineer in the United States, Canada and Mexico whose address was given in *Poor's Manual*. To these chief engineers there was also sent another circular-letter with a ballot that read as follows:

"'I $\left\{\begin{matrix}\text{agree}\\ \text{do not agree}\end{matrix}\right\}$ to use the "Compromise Standard System of Live Loads for Railway Bridges" when calling for bids on railroad-bridge work, or when having plans prepared for railroad bridges.'

"'I $\left\{\begin{matrix}\text{agree}\\ \text{do not agree}\end{matrix}\right\}$ to specify that the "Equivalent Uniform-Load Method" is to be used in computing stresses in the bridges that are to be designed for my road.'

"Over one hundred chief engineers thus addressed voted in favor of both propositions, and very few were opposed."

The Compromise Standard System of Live Loads for Railroad Bridges has now been in use for six years, and all who have used it and its equivalents endorse heartily both the loads themselves and the equivalent method of calculating stresses.

Unfortunately, locomotives and cars have both grown somewhat heavier since the writer's pamphlet was issued, so that it is now neces-

Mr. Waddell. sary to add to the live loads "Class *S*," for which the axle loads would be as follows:

Pilot axle	22 000 lbs.
Driving axles	43 000 lbs. each.
Tender axles	25 000 lbs. each.
Cars	4 400 lbs. per lineal foot.
Alternative axle load	54 000 lbs. each, on two axles only.

The equivalent curves can be drawn on the diagrams by eye, correctly enough for all practical purposes, by simply extending upward the spacing of the curves.

There is one point in connection with all these equivalent curves to which the writer desires to call attention, viz., the equivalent loads which they represent were figured for cars preceding as well as following the locomotives, in the case of chord stresses.

The writer desires also to call attention to the short method, evolved by him some eight years ago, of computing for any truss, by one simple graphic diagram and by the use of a slide rule, all the main-member stresses, both direct and reverse.

The method is as follows:

Assume that there is an upward reaction at one end of the span equal to 100 000 lbs., caused by an unknown load placed at the first panel point from the other end of the span, then find graphically the stress in each web-member from end to end of span, caused by this assumed upward reaction. Then calculate the value of the live-load reaction for the maximum stress in each web-member by means of the slide rule and the following formula and table in which n is the number of panels in the span, n' is the number of the panel point at the head of the train, counting from the loaded end of the span, and C is the coefficient of $\frac{L}{n}$.

Live-load reaction for the head of train at $n' = C \times \frac{L}{n}$.

n'	C	n'	C	n'	C	n'	C
1	1	7	28	13	91	19	190
2	3	8	36	14	105	20	210
3	6	9	45	15	120	21	231
4	10	10	55	16	136	22	253
5	15	11	66	17	153	23	276
6	21	12	78	18	171	24	300

Then, still using the slide rule, find the greatest live-load stress in each web-member by the following equation:

$$\text{Stress required} = \text{Stress from Assumed Reaction} \times \frac{\text{Actual Reaction}}{\text{Assumed Reaction}}$$

Where the panels are divided, as in the Petit truss, and where

inclined sub-posts are used, the tensile stress in the upper half of each main diagonal thus found will have to be corrected by subtracting therefrom a stress equal to $\frac{L}{2}$ Sec. A, where A is the inclination of the diagonal to the vertical. But when inclined sub-ties are used instead of inclined sub-posts, the correction just referred to will apply only to the compressive stresses in the lower halves of the main diagonals. The reason for making this correction, as will be at once evident to any one who is accustomed to finding stresses in Petit trusses, is that the method above outlined ignores the subdivision of the panels when ascertaining by graphics the stresses caused by the assumed upward reaction. Mr. Waddell.

The writer's sole reason for again bringing forward this quick method is that he would like bridge engineers to learn to use it and thus save themselves much valuable time and energy, as it cuts down the work of computing web stresses to a very small percentage of the amount usually expended thereon.

In conclusion, it appears from the opinions advanced in the oral discussion, that the time has come to drop from engineering practice and to relegate to text books, where it can still serve as an excellent example of mental gymnastics, the pseudo-exact, laborious and tedious method of Wheel Concentrations, and to adopt instead some simpler methods; consequently, the writer herewith proposes that the "Compromise Standard System of Live Loads for Railroad Bridges" and the "Equivalent Uniform-Load Method" of computing stresses be adopted by the profession for general practice.

O. E. Selby, Jun. Am. Soc. C. E. (by letter).—It is the purpose of the writer to present, first, three tables giving actual moments and shears in trusses and plate-girders produced by the loading in "Cooper's 1896 Specifications;" second, a table of the single-excess loads which, with the constant uniform train-load, will produce the same moments and shears as the wheel-loads; third, two diagrams on which these excess loads are plotted in order to determine their laws of variation; fourth, three simple formulas deduced from the diagrams, which will give values of excess loads producing moments and shears substantially equal to those produced by the wheel loads; fifth, a simple specification for loading which will combine in one operation the computing of the quiescent stresses and the allowance for impact. Mr. Selby.

The tables for moments and shears for Cooper's "$E40$" have been computed systematically and checked carefully, and will be useful to those who still prefer to, or find it necessary to, compute stresses by the actual wheel-loads for this specification.

Table No. 10 gives the moment, end shear, center shear and shear at quarter point for plate-girder spans, varying by increments of 1 ft., from 10 to 125 ft. Tables Nos. 11 and 12, respectively, give moments and shears for truss spans of 5, 6, 7, 8 and 9 panels, and panel lengths

Mr. Selby. varying by increments of 1 ft. from 20 to 35 ft. This includes span lengths between 100 ft. and 315 ft. The shears apply only to trusses with parallel chords. The arrangement of the tabulation by panel lengths rather than span lengths is believed to be new, and facilitates a complete tabulation in a small space. Values for intermediate panel lengths can be obtained by direct interpolation. All values in the tables are for one truss or girder.

Having prepared the foregoing tables for his own use, some time ago, the writer began to cast about for a simple substitute loading which could be used instead of wheel loads in general without material variation in the results, and for some rule for varying the substitute loading to correspond with different engine loadings. He made many comparative computations using various single excesses, double excesses, uniform loads, etc., with results in favor of the single-excess method, but without finding any one excess load which gave satisfactory results for widely different span lengths. Finally, it occurred to him to compute, for the various positions and span lengths, the excess loads which, with the uniform train-load, would produce the actual moments and shears in the table, and to study their laws of variation.

Table No. 13 gives the values of such excess loads E, for one truss or girder, for all panel points of trusses of 5, 6, 7, 8 and 9 panels, with panel lengths varying by 3 ft. from 20 to 35 ft., and for plate-girders of lengths varying by 5 ft. from 10 to 125 ft.

The formulas for computing E are very simple. Let M = total actual moment, M_u = moment from uniform load and M_e = moment from excess load. Then $M = M_u + M_e$. Similarly for shears $S = S_u + S_e$.

Let w = uniform train-load per foot and l = span.

For moment in a plate-girder:

$$M = \frac{1}{8} w l^2 + \frac{1}{4} E l, \text{ from which}$$

$$E = \frac{4 M}{l} - \frac{w}{2} l.$$

For a train-load of 4 000 lbs. per foot, with 1 000-lb. units, $w = 2$ for one girder, then $E = \frac{4 M}{l} - l.$

For end shear in a plate-girder:

$$S = \frac{1}{2} w l + E, \text{ from which } E = S - \frac{1}{2} w l.$$

If $w = 2$, $E = S - l$.

For center shear in a plate-girder:

$$S = \frac{1}{4} w \frac{l}{2} + \frac{E}{2} = \frac{1}{8} w l + \frac{E}{2}, \text{ from which}$$

$$E = 2 S - \frac{1}{4} w l.$$

If $w = 2$, $E = 2 S - \frac{1}{2} l.$

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For moments in a truss, let N = whole number of panels and n = number of panels to the left of the point in question; then, considering the uniform panel load concentrated at the panel point,

$$M_u = \frac{N-1}{2}\, p\, w \times n\, p - \frac{n\,(n-1)}{2}\, p \times p\, w = p^2\, \frac{w}{2}\, n\,(N-n).$$

If $w = 2$, $M_u = p^2\, n\,(N-n)$.

$$M_e = \frac{N-n}{N}\, E\, n\, p$$

$$M = p^2\, n\,(N-n) + \frac{N-n}{N}\, E\, n\, p = n\,(N-n)\, p\left(p + \frac{E}{N}\right), \text{ from}$$

which

$$E = N\left(\frac{M}{n\,(N-n)\,p} - p\right)$$

For shears in a truss:

$$S_u = p\, w\, \frac{(N-n+1)\,(N-n)}{2\,N}.$$

If $w = 2$, $S_u = p\, \frac{(N-n+1)\,(N-n)}{N}$.

$$S_e = \frac{N-n}{N}\, E.$$

$$S = p\, \frac{(N-n+1)\,(N-n)}{N} + \frac{N-n}{N}\, E, \text{ from which}$$

$$E = \frac{N}{N-n}\, S - p\,(N-n+1).$$

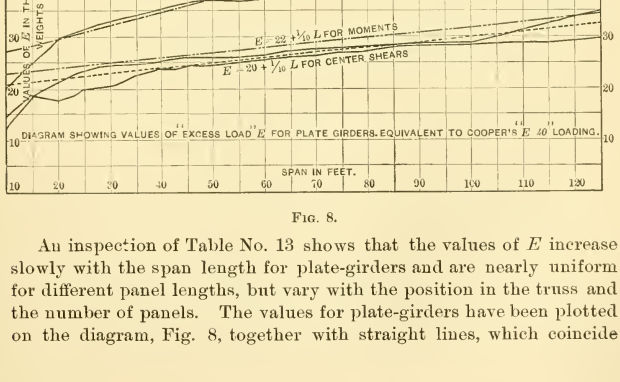

Fig. 8.

An inspection of Table No. 13 shows that the values of E increase slowly with the span length for plate-girders and are nearly uniform for different panel lengths, but vary with the position in the truss and the number of panels. The values for plate-girders have been plotted on the diagram, Fig. 8, together with straight lines, which coincide

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TABLE No. 10.—Maximum Moments and Shears in Plate-Girders. Cooper's 1896 Specifications "E 40."

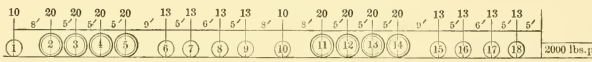
Moments in Thousands of Ft.-Lbs. Shears in Thousands of Lbs. Weights on One Rail.

Span.	Moment.	Shears. End.	Shears. ¼ P't.	Shears. Center.	Span.	Moment.	Shears. End.	Shears. ¼ P't.	Shears. Center.	Span.	Moment.	Shears. End.	Shears. ¼ P't.	Shears. Center.
1	0.0	10.0	10.0	10.0	36	548.6	70.6	44.1	29.4	66	1 539.5	105.0	63.6	29.7
2	37.8	10.0	10.0	10.0	37	574.3	71.9	44.8	29.4	67	1 581.5	106.4	64.2	30.0
3	75.5	10.0	10.0	10.0	38	600.0	73.1	45.4	29.4	68	1 623.5	107.8	64.8	30.0
4	113.3	10.0	10.0	10.0	39	626.6	74.3	46.0	29.4	69	1 665.5	109.2	65.4	30.0
5	151.0	10.0	10.0	10.0	40	653.2	75.4	46.6	29.4	70	1 706.9	110.6	65.9	30.1
6	188.8	10.0	10.9	10.0	41	679.8	76.6	47.2	29.4	71	1 749.3	111.9	66.5	30.1
7	226.5	10.9	11.7	10.0	42	706.4	77.8	47.8	29.4	72	1 791.7	113.3	67.0	30.1
8	264.3	11.7	12.3	10.0	43	733.0	78.9	48.4	29.4	73	1 834.1	114.7	67.6	30.1
9	302.0	12.3	12.9	10.0	44	759.6	80.1	48.9	29.4	74	1 876.5	116.1	68.2	30.1
10	339.8	12.9	13.3	10.0	45	786.2	81.2	49.5	29.4	75	1 918.9	117.5	68.8	30.1
11	377.5	13.3	13.8	10.0	46	812.8	82.4	50.1	29.4	76	1 961.4	118.9	69.4	30.1
12	415.3	13.8	14.2	10.0	47	839.4	83.5	50.7	29.4	77	2 003.8	120.3	69.9	30.1
13	453.0	14.2	14.7	10.0	48	866.0	84.7	51.3	29.4	78	2 046.2	121.7	70.5	30.1
14	490.8	14.7	15.2	10.0	49	892.6	85.8	51.8	29.4	79	2 088.6	123.0	71.1	30.1
15	528.5	15.2	15.6	10.0	50	919.2	87.0	52.4	29.4	80	2 131.0	124.4	71.7	30.1
16	566.3	15.6	16.1	10.0	51	945.8	88.2	53.0	29.4	81	2 173.4	125.8	72.3	30.1
17	604.0	16.1	16.5	10.0	52	972.4	89.3	53.6	29.4	82	2 215.8	127.2	72.8	30.1
18	641.8	16.5	16.9	10.0	53	999.0	90.5	54.2	29.4	83	2 258.2	128.6	73.4	30.1
19	679.5	16.9	17.4	10.0	54	1 025.6	91.6	54.8	29.4	84	2 300.6	129.9	74.0	30.1
20	717.3	17.4	17.8	10.0	55	1 052.2	92.8	55.4	29.4	85	2 343.0	131.3	74.6	30.1
21	755.0	17.8	18.2	10.0	56	1 078.8	94.0	56.0	29.4	86	2 385.4	132.7	75.1	30.1
22	792.8	18.2	18.7	10.0	57	1 105.4	95.1	56.6	29.4	87	2 427.8	134.1	75.7	30.1
23	830.5	18.7	19.1	10.0	58	1 132.0	96.3	57.2	29.4	88	2 470.2	135.5	76.3	30.1
24	868.3	19.1	19.5	10.0	59	1 158.6	97.4	57.8	29.4	89	2 512.6	136.8	76.9	30.1
25	906.0	19.5	20.0	10.0	60	1 185.2	98.6	58.3	29.4	90	2 555.0	138.2	77.4	30.1
26	943.8	20.0	20.4	10.0	61	1 211.8	99.8	58.9	29.4	91	2 597.4	139.6	78.0	30.1
27	981.5	20.4	20.8	10.0	62	1 238.4	100.9	59.5	29.4	92	2 639.8	141.0	78.6	30.1
28	1 019.3	20.8	21.3	10.0	63	1 265.0	102.1	60.1	29.4	93	2 682.2	142.4	79.2	30.1
29	1 057.0	21.3	21.7	10.0	64	1 291.6	103.2	60.7	29.4	94	2 724.6	143.8	79.7	30.1
30	1 094.8	21.7	22.1	10.0	65	1 318.2	104.4	61.3	29.4	95	2 767.0	145.2	80.3	30.1
31	1 132.5	22.1	22.6	10.0						96	2 809.4	146.6	80.9	30.1
32	1 170.3	22.6	23.0	10.0						97	2 851.8	148.0	81.5	30.1
33	1 208.0	23.0	23.4	10.0						98	2 894.2	149.4	82.0	30.1
34	1 245.8	23.4	23.9	10.0						99	2 936.6	150.8	82.6	30.1
35	1 283.5	23.9	24.3	10.0						100	2 979.0	152.2	83.2	30.1

Span.	Moment.	Shears. End.	Shears. ¼ P't.	Shears. Center.
1	0.0	10.0	10.0	10.0
2	37.8	10.0	10.0	10.0
3	75.5	10.0	10.0	10.0
4	113.3	10.0	10.0	10.0
5	151.0	10.0	10.0	10.0
6	188.8	10.0	10.9	10.0
7	226.5	10.9	11.7	10.0
8	264.3	11.7	12.3	10.0
9	302.0	12.3	12.9	10.0
10	339.8	12.9	13.3	10.0
11	377.5	13.3	13.8	10.0
12	415.3	13.8	14.2	10.0
13	453.0	14.2	14.7	10.0
14	490.8	14.7	15.2	10.0
15	528.5	15.2	15.6	10.0
16	566.3	15.6	16.1	10.0
17	604.0	16.1	16.5	10.0
18	641.8	16.5	16.9	10.0
19	679.5	16.9	17.4	10.0
20	717.3	17.4	17.8	10.0
21	755.0	17.8	18.2	10.0
22	792.8	18.2	18.7	10.0
23	830.5	18.7	19.1	10.0
24	868.3	19.1	19.5	10.0
25	906.0	19.5	20.0	10.0
26	943.8	20.0	20.4	10.0
27	981.5	20.4	20.8	10.0
28	1 019.3	20.8	21.3	10.0
29	1 057.0	21.3	21.7	10.0
30	1 094.8	21.7	22.1	10.0
31	1 132.5	22.1	22.6	10.0
32	1 170.3	22.6	23.0	10.0
33	1 208.0	23.0	23.4	10.0
34	1 245.8	23.4	23.9	10.0
35	1 283.5	23.9	24.3	10.0
36	1 321.3	24.3	24.7	10.0
37	1 359.0	24.7	25.2	10.0
38	1 396.8	25.2	25.6	10.0
39	1 434.5	25.6	26.0	10.0
40	1 472.3	26.0	26.5	10.0
41	1 510.0	26.5	26.9	10.0
42	1 547.8	26.9	27.3	10.0
43	1 585.5	27.3	27.8	10.0
44	1 623.3	27.8	28.2	10.0
45	1 661.0	28.2	28.6	10.0
46	1 698.8	28.6	29.1	10.0
47	1 736.5	29.1	29.5	10.0
48	1 774.3	29.5	29.9	10.0
49	1 812.0	29.9	30.4	10.0
50	1 849.8	30.4	30.8	10.0
51	1 887.5	30.8	31.2	10.0
52	1 925.3	31.2	31.7	10.0
53	1 963.0	31.7	32.1	10.0
54	2 000.8	32.1	32.5	10.0
55	2 038.5	32.5	33.0	10.0
56	2 076.3	33.0	33.4	10.0
57	2 114.0	33.4	33.8	10.0
58	2 151.8	33.8	34.3	10.0
59	2 189.5	34.3	34.7	10.0
60	2 227.3	34.7	35.1	10.0
61	2 265.0	35.1	35.6	10.0
62	2 302.8	35.6	36.0	10.0
63	2 340.5	36.0	36.4	10.0
64	2 378.3	36.4	36.9	10.0
65	2 416.0	36.9	37.3	10.0
66	2 453.8	37.3	37.7	10.0
67	2 491.5	37.7	38.2	10.0
68	2 529.3	38.2	38.6	10.0
69	2 567.0	38.6	39.0	10.0
70	2 604.8	39.0	39.5	10.0
71	2 642.5	39.5	39.9	10.0
72	2 680.3	39.9	40.3	10.0
73	2 718.0	40.3	40.8	10.0
74	2 755.8	40.8	41.2	10.0
75	2 793.5	41.2	41.6	10.0
76	2 831.3	41.6	42.1	10.0
77	2 869.0	42.1	42.5	10.0
78	2 906.8	42.5	42.9	10.0
79	2 944.5	42.9	43.4	10.0
80	2 982.3	43.4	43.8	10.0
81	3 020.0	43.8	44.2	10.0
82	3 057.8	44.2	44.7	10.0
83	3 095.5	44.7	45.1	10.0
84	3 133.3	45.1	45.5	10.0
85	3 171.0	45.5	46.0	10.0
86	3 208.8	46.0	46.4	10.0
87	3 246.5	46.4	46.8	10.0
88	3 284.3	46.8	47.3	10.0
89	3 322.0	47.3	47.7	10.0
90	3 359.8	47.7	48.1	10.0
91	3 397.5	48.1	48.6	10.0
92	3 435.3	48.6	49.0	10.0
93	3 473.0	49.0	49.4	10.0
94	3 510.8	49.4	49.9	10.0
95	3 548.5	49.9	50.3	10.0
96	3 586.3	50.3	50.7	10.0
97	3 624.0	50.7	51.2	10.0
98	3 661.8	51.2	51.6	10.0
99	3 699.5	51.6	52.0	10.0
100	3 737.3	52.0	52.5	10.0
101	3 775.0	52.5	52.9	10.0
102	3 812.8	52.9	53.3	10.0
103	3 850.5	53.3	53.8	10.0
104	3 888.3	53.8	54.2	10.0
105	3 926.0	54.2	54.6	10.0
106	3 963.8	54.6	55.1	10.0
107	4 001.5	55.1	55.5	10.0
108	4 039.3	55.5	55.9	10.0
109	4 077.0	55.9	56.4	10.0
110	4 114.8	56.4	56.8	10.0
111	4 152.5	56.8	57.2	10.0
112	4 190.3	57.2	57.7	10.0
113	4 228.0	57.7	58.1	10.0
114	4 265.8	58.1	58.5	10.0
115	4 303.5	58.5	59.0	10.0
116	4 341.3	59.0	59.4	10.0
117	4 379.0	59.4	59.8	10.0
118	4 416.8	59.8	60.3	10.0
119	4 454.5	60.3	60.7	10.0
120	4 492.3	60.7	61.1	10.0
121	4 530.0	61.1	61.6	10.0
122	4 567.8	61.6	62.0	10.0
123	4 605.5	62.0	62.4	10.0
124	4 643.3	62.4	62.9	10.0
125	4 681.0	62.9	63.3	10.0
126	4 718.8	63.3	63.7	10.0
127	4 756.5	63.7	64.2	10.0
128	4 794.3	64.2	64.6	10.0
129	4 832.0	64.6	65.0	10.0
130	4 869.8	65.0	65.5	10.0
131	4 907.5	65.5	65.9	10.0
132	4 945.3	65.9	66.3	10.0
133	4 983.0	66.3	66.8	10.0
134	5 020.8	66.8	67.2	10.0
135	5 058.5	67.2	67.6	10.0
136	5 096.3	67.6	68.1	10.0
137	5 134.0	68.1	68.5	10.0
138	5 171.8	68.5	68.9	10.0
139	5 209.5	68.9	69.4	10.0
140	5 247.3	69.4	69.8	10.0
141	5 285.0	69.8	70.2	10.0
142	5 322.8	70.2	70.7	10.0
143	5 360.5	70.7	71.1	10.0
144	5 398.3	71.1	71.5	10.0
145	5 436.0	71.5	72.0	10.0
146	5 473.8	72.0	72.4	10.0
147	5 511.5	72.4	72.8	10.0
148	5 549.3	72.8	73.3	10.0
149	5 587.0	73.3	73.7	10.0
150	5 624.8	73.7	74.1	10.0
151	5 662.5	74.1	74.6	10.0
152	5 700.3	74.6	75.0	10.0
153	5 738.0	75.0	75.4	10.0
154	5 775.8	75.4	75.9	10.0
155	5 813.5	75.9	76.3	10.0
156	5 851.3	76.3	76.7	10.0
157	5 889.0	76.7	77.2	10.0
158	5 926.8	77.2	77.6	10.0
159	5 964.5	77.6	78.0	10.0
160	6 002.3	78.0	78.5	10.0
161	6 040.0	78.5	78.9	10.0
162	6 077.8	78.9	79.3	10.0
163	6 115.5	79.3	79.8	10.0
164	6 153.3	79.8	80.2	10.0
165	6 191.0	80.2	80.6	10.0
166	6 228.8	80.6	81.1	10.0
167	6 266.5	81.1	81.5	10.0
168	6 304.3	81.5	81.9	10.0
169	6 342.0	81.9	82.4	10.0
170	6 379.8	82.4	82.8	10.0
171	6 417.5	82.8	83.2	10.0
172	6 455.3	83.2	83.7	10.0
173	6 493.0	83.7	84.1	10.0
174	6 530.8	84.1	84.5	10.0
175	6 568.5	84.5	85.0	10.0
176	6 606.3	85.0	85.4	10.0
177	6 644.0	85.4	85.8	10.0
178	6 681.8	85.8	86.3	10.0
179	6 719.5	86.3	86.7	10.0
180	6 757.3	86.7	87.1	10.0
181	6 795.0	87.1	87.6	10.0
182	6 832.8	87.6	88.0	10.0
183	6 870.5	88.0	88.4	10.0
184	6 908.3	88.4	88.9	10.0
185	6 946.0	88.9	89.3	10.0
186	6 983.8	89.3	89.7	10.0
187	7 021.5	89.7	90.2	10.0
188	7 059.3	90.2	90.6	10.0
189	7 097.0	90.6	91.0	10.0
190	7 134.8	91.0	91.5	10.0
191	7 172.5	91.5	91.9	10.0
192	7 210.3	91.9	92.3	10.0
193	7 248.0	92.3	92.8	10.0
194	7 285.8	92.8	93.2	10.0
195	7 323.5	93.2	93.6	10.0
196	7 361.3	93.6	94.1	10.0
197	7 399.0	94.1	94.5	10.0
198	7 436.8	94.5	94.9	10.0
199	7 474.5	94.9	95.4	10.0
200	7 512.3	95.4	95.8	10.0

Span.	Moment.	Shears. End.	Shears. ¼ P't.	Shears. Center.
2	998.9	144.8	83.3	38.3
3	049.9	146.2	83.9	38.3
4	100.9	147.6	84.5	38.3
5	151.9	149.0	85.0	39.4
6	202.9	150.4	85.6	39.4
7	253.9	151.8	86.2	39.4
8	304.9	153.2	86.8	39.4
9	355.9	154.6	87.3	39.9
10	406.9	156.0	87.9	40.1
11	457.9	157.4	88.5	40.4
12	508.9	158.8	89.1	40.6
13	559.9	160.2	89.6	40.9
14	610.9	161.6	90.2	40.9
15	661.9	163.0	90.8	41.1
16	712.9	164.4	91.4	41.3
17	763.9	165.8	91.9	41.8
18	814.9	167.2	92.5	42.0
19	865.9	168.6	93.1	42.0
20	916.9	170.0	93.6	42.0
21	967.9	171.4	94.2	42.5
22	1 018.9	172.8	94.8	42.5
23	1 069.9	174.2	95.3	42.5
24	1 120.9	175.6	95.9	43.1
25	1 171.9	177.0	96.5	43.1
26	1 222.9	178.4	97.0	43.4
27	1 273.9	179.8	97.6	43.6
28	1 324.9	181.2	98.2	43.9
29	1 375.9	182.6	98.7	44.1
30	1 426.9	184.0	99.3	44.6
31	1 477.9	185.4	99.9	44.6
32	1 528.9	186.8	100.4	44.8
33	1 579.9	188.2	101.0	45.3
34	1 630.9	189.6	101.6	45.3
35	1 681.9	191.0	102.1	45.6
36	1 732.9	192.4	102.7	45.6
37	1 783.9	193.8	103.3	45.8
38	1 834.9	195.2	103.8	46.1
39	1 885.9	196.6	104.4	46.3
40	1 936.9	198.0	105.0	46.6
41	1 987.9	199.4	105.5	46.8
42	2 038.9	200.8	106.1	47.1
43	2 089.9	202.2	106.7	47.3
44	2 140.9	203.6	107.2	47.6
45	2 191.9	205.0	107.8	47.8
46	2 242.9	206.4	108.4	48.1
47	2 293.9	207.8	108.9	48.3
48	2 344.9	209.2	109.5	48.6
49	2 395.9	210.6	110.1	48.8
50	2 446.9	212.0	110.6	49.1
51	2 497.9	213.4	111.2	49.3
52	2 548.9	214.8	111.8	49.6
53	2 599.9	216.2	112.3	49.8
54	2 650.9	217.6	112.9	50.1
55	2 701.9	219.0	113.5	50.3
56	2 752.9	220.4	114.0	50.6
57	2 803.9	221.8	114.6	50.8
58	2 854.9	223.2	115.2	51.1
59	2 905.9	224.6	115.7	51.3
60	2 956.9	226.0	116.3	51.6
61	3 007.9	227.4	116.9	51.8
62	3 058.9	228.8	117.4	52.1
63	3 109.9	230.2	118.0	52.3
64	3 160.9	231.6	118.6	52.6
65	3 211.9	233.0	119.1	52.8
66	3 262.9	234.4	119.7	53.1
67	3 313.9	235.8	120.3	53.3
68	3 364.9	237.2	120.8	53.6
69	3 415.9	238.6	121.4	53.8
70	3 466.9	240.0	122.0	54.1
71	3 517.9	241.4	122.5	54.3
72	3 568.9	242.8	123.1	54.6
73	3 619.9	244.2	123.7	54.8
74	3 670.9	245.6	124.2	55.1
75	3 721.9	247.0	124.8	55.3
76	3 772.9	248.4	125.4	55.6
77	3 823.9	249.8	125.9	55.8
78	3 874.9	251.2	126.5	56.1
79	3 925.9	252.6	127.1	56.3
80	3 976.9	254.0	127.6	56.6
81	4 027.9	255.4	128.2	56.8
82	4 078.9	256.8	128.8	57.1
83	4 129.9	258.2	129.3	57.3
84	4 180.9	259.6	129.9	57.6
85	4 231.9	261.0	130.5	57.8
86	4 282.9	262.4	131.0	58.1
87	4 333.9	263.8	131.6	58.3
88	4 384.9	265.2	132.2	58.6
89	4 435.9	266.6	132.7	58.8
90	4 486.9	268.0	133.3	59.1
91	4 537.9	269.4	133.9	59.3
92	4 588.9	270.8	134.4	59.6
93	4 639.9	272.2	135.0	59.8
94	4 690.9	273.6	135.6	60.1
95	4 741.9	275.0	136.1	60.3
96	4 792.9	276.4	136.7	60.6
97	4 843.9	277.8	137.3	60.8
98	4 894.9	279.2	137.8	61.1
99	4 945.9	280.6	138.4	61.3
100	4 996.9	282.0	139.0	61.6
101	5 047.9	283.4	139.5	61.8
102	5 098.9	284.8	140.1	62.1
103	5 149.9	286.2	140.7	62.3
104	5 200.9	287.6	141.2	62.6
105	5 251.9	289.0	141.8	62.8
106	5 302.9	290.4	142.4	63.1
107	5 353.9	291.8	142.9	63.3
108	5 404.9	293.2	143.5	63.6
109	5 455.9	294.6	144.1	63.8
110	5 506.9	296.0	144.6	64.1
111	5 557.9	297.4	145.2	64.3
112	5 608.9	298.8	145.8	64.6
113	5 659.9	300.2	146.3	64.8
114	5 710.9	301.6	146.9	65.1
115	5 761.9	303.0	147.5	65.3
116	5 812.9	304.4	148.0	65.6
117	5 863.9	305.8	148.6	65.8
118	5 914.9	307.2	149.2	66.1
119	5 965.9	308.6	149.7	66.3
120	6 016.9	310.0	150.3	66.6
121	6 067.9	311.4	150.9	66.8
122	6 118.9	312.8	151.4	67.1
123	6 169.9	314.2	152.0	67.3
124	6 220.9	315.6	152.6	67.6
125	6 271.9	317.0	153.1	67.8
126	6 322.9	318.4	153.7	68.1
127	6 373.9	319.8	154.3	68.3
128	6 424.9	321.2	154.8	68.6
129	6 475.9	322.6	155.4	68.8
130	6 526.9	324.0	156.0	69.1
131	6 577.9	325.4	156.5	69.3
132	6 628.9	326.8	157.1	69.6
133	6 679.9	328.2	157.7	69.8
134	6 730.9	329.6	158.2	70.1
135	6 781.9	331.0	158.8	70.3
136	6 832.9	332.4	159.4	70.6
137	6 883.9	333.8	159.9	70.8
138	6 934.9	335.2	160.5	71.1
139	6 985.9	336.6	161.1	71.3
140	7 036.9	338.0	161.6	71.6
141	7 087.9	339.4	162.2	71.8
142	7 138.9	340.8	162.8	72.1
143	7 189.9	342.2	163.3	72.3
144	7 240.9	343.6	163.9	72.6
145	7 291.9	345.0	164.5	72.8
146	7 342.9	346.4	165.0	73.1
147	7 393.9	347.8	165.6	73.3
148	7 444.9	349.2	166.2	73.6
149	7 495.9	350.6	166.7	73.8
150	7 546.9	352.0	167.3	74.1
151	7 597.9	353.4	167.9	74.3
152	7 648.9	354.8	168.4	74.6
153	7 699.9	356.2	169.0	74.8
154	7 750.9	357.6	169.6	75.1
155	7 801.9	359.0	170.1	75.3
156	7 852.9	360.4	170.7	75.6
157	7 903.9	361.8	171.3	75.8
158	7 954.9	363.2	171.8	76.1
159	8 005.9	364.6	172.4	76.3
160	8 056.9	366.0	173.0	76.6
161	8 107.9	367.4	173.5	76.8
162	8 158.9	368.8	174.1	77.1
163	8 209.9	370.2	174.7	77.3
164	8 260.9	371.6	175.2	77.6
165	8 311.9	373.0	175.8	77.8
166	8 362.9	374.4	176.4	78.1
167	8 413.9	375.8	176.9	78.3
168	8 464.9	377.2	177.5	78.6
169	8 515.9	378.6	178.1	78.8
170	8 566.9	380.0	178.6	79.1
171	8 617.9	381.4	179.2	79.3
172	8 668.9	382.8	179.8	79.6
173	8 719.9	384.2	180.3	79.8
174	8 770.9	385.6	180.9	80.1
175	8 821.9	387.0	181.5	80.3
176	8 872.9	388.4	182.0	80.6
177	8 923.9	389.8	182.6	80.8
178	8 974.9	391.2	183.2	81.1
179	9 025.9	392.6	183.7	81.3
180	9 076.9	394.0	184.3	81.6
181	9 127.9	395.4	184.9	81.8
182	9 178.9	396.8	185.4	82.1
183	9 229.9	398.2	186.0	82.3
184	9 280.9	399.6	186.6	82.6
185	9 331.9	401.0	187.1	82.8
186	9 382.9	402.4	187.7	83.1
187	9 433.9	403.8	188.3	83.3
188	9 484.9	405.2	188.8	83.6
189	9 535.9	406.6	189.4	83.8
190	9 586.9	408.0	190.0	84.1
191	9 637.9	409.4	190.5	84.3
192	9 688.9	410.8	191.1	84.6
193	9 739.9

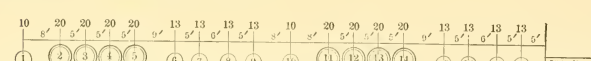
TABLE No. 11.—Maximum Moments. Cooper's 1896 Specification "E 40." Moments in Thousands of Foot-Pounds. Weights on One Rail.



Panel Points.	Panel Lengths.															
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
1/5	2 186.2	2 400.4	2 622.4	2 849.8	3 081.8	3 320.2	3 565.0	3 816.2	4 073.8	4 337.8	4 608.2	4 890.0	5 181.2	5 478.8	5 782.8	6 093.2
1/6	2 689.3	2 942.7	3 206.8	3 479.3	3 760.2	4 049.3	4 346.8	4 652.7	4 966.8	5 289.3	5 620.2	5 959.3	6 312.7	6 676.8	7 049.3	7 430.2
1/7	3 163.7	3 453.4	3 761.3	4 080.1	4 409.3	4 748.7	5 098.4	5 458.4	5 828.7	6 209.3	6 600.1	7 001.3	7 413.4	7 843.7	8 284.3	8 735.1
1/8	3 619.5	3 951.1	4 298.1	4 663.0	5 040.1	5 429.5	5 831.1	6 245.0	6 671.1	7 109.5	7 560.1	8 023.0	8 498.1	8 991.1	9 499.5	10 020.1
1/9	4 062.9	4 436.2	4 823.8	5 233.9	5 658.8	6 097.9	6 551.2	7 018.8	7 500.6	7 996.6	8 506.8	9 031.2	9 569.9	10 125.6	10 701.6	11 291.8
2/5	3 154.4	3 477.6	3 813.6	4 154.4	4 512.0	4 874.4	5 241.6	5 613.6	5 993.4	6 387.0	6 785.4	7 188.6	7 596.6	8 009.4	8 472.4	8 953.6
2/6	4 204.7	4 599.3	4 999.3	5 404.7	5 815.3	6 234.7	6 682.3	7 167.7	7 663.7	8 173.7	8 704.3	9 245.7	9 797.7	10 381.7	10 981.7	11 598.3
2/7	5 060.6	5 531.4	6 024.3	6 537.9	7 073.6	7 623.6	8 187.9	8 783.6	9 393.6	10 017.9	10 664.3	11 331.4	12 012.9	12 715.4	13 448.3	14 204.6
2/8	5 966.3	6 530.3	7 112.3	7 712.3	8 344.0	8 995.0	9 664.0	10 360.0	11 083.0	11 824.0	12 584.3	13 376.3	14 186.3	15 014.3	15 880.3	16 768.3
2/9	6 862.4	7 503.1	8 173.6	8 865.8	9 588.1	10 339.4	11 112.6	11 910.1	12 745.4	13 602.6	14 481.4	15 393.8	16 331.8	17 291.6	18 285.6	19 313.8
3/6	4 596.5	5 057.0	5 536.5	6 030.0	6 532.5	7 056.5	7 592.0	8 136.5	8 690.0	9 252.5	9 836.5	10 432.0	11 036.5	11 650.0	12 272.5	12 916.5
3/7	6 117.1	6 677.7	7 262.9	7 861.1	8 487.3	9 155.9	9 850.3	10 563.4	11 290.3	12 030.9	12 787.3	13 572.4	14 371.3	15 183.9	16 010.1	16 859.4
3/8	7 387.6	8 056.5	8 795.4	9 581.0	10 385.4	11 208.5	12 051.0	12 930.4	13 828.5	14 745.4	15 681.0	16 648.5	17 640.4	18 651.0	19 680.4	20 735.4
3/9	8 704.3	9 520.3	10 381.7	11 293.7	12 245.7	13 221.7	14 221.7	15 257.3	16 325.3	17 417.3	18 533.3	19 681.7	20 865.7	22 073.7	23 305.7	24 566.7
4/8	7 954.0	8 690.0	9 444.5	10 232.5	11 036.5	11 856.5	12 700.0	13 572.0	14 460.0	15 368.5	16 324.5	17 309.0	18 317.0	19 359.0	20 423.0	21 520.5
4/9	9 738.2	10 647.1	11 578.2	12 547.1	13 544.9	14 564.9	15 611.6	16 698.2	17 807.1	18 942.2	20 135.6	21 363.9	22 623.9	23 926.1	25 257.2	26 631.1

TABLE No. 12.—Maximum Shears.

Cooper's 1896 Specification "E 40."
Shears in Thousands of Pounds.
Weights on One Rail.



Panel Points.	Panel Lengths.															
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
1/5	109.3	114.3	119.2	123.9	128.4	132.8	137.1	141.3	145.5	149.6	153.6	157.7	161.9	166.0	170.1	174.1
1/6	134.5	140.1	145.8	151.3	156.7	162.0	167.2	172.3	177.4	182.4	187.3	192.2	197.3	202.3	207.3	212.3
1/7	158.2	164.4	171.0	177.4	183.7	189.9	196.1	202.2	208.2	214.1	220.0	225.8	231.7	237.7	243.7	249.6
1/8	181.0	188.1	195.4	202.7	210.0	217.2	224.3	231.3	238.3	245.2	252.0	258.8	265.6	272.5	279.4	286.3
1/9	203.1	211.2	219.3	227.6	235.8	243.9	252.0	260.0	267.9	275.7	283.6	291.3	299.1	306.8	314.8	322.6
2/5	63.3	66.1	68.9	71.9	74.8	77.3	80.1	82.9	85.7	88.5	91.2	93.8	96.3	98.8	101.2	103.6
2/6	88.8	92.8	96.6	100.3	104.1	107.6	111.2	114.8	118.4	121.9	125.3	128.7	132.1	135.4	138.7	141.9
2/7	113.6	118.3	122.9	127.4	131.8	136.2	140.7	145.2	149.6	154.0	158.3	162.6	166.8	171.0	175.1	179.2
2/8	137.0	142.5	148.0	153.3	158.6	163.9	169.3	174.7	180.0	185.3	190.5	195.7	200.8	205.9	211.0	216.0
2/9	159.6	166.0	172.4	178.6	184.8	191.0	197.3	203.6	209.9	216.1	222.2	228.3	234.4	240.4	246.4	252.4
3/5	31.0	32.2	33.3	34.3	35.4	36.4	37.5	38.6	39.7	41.0	42.2	43.4	44.7	45.9	47.3	48.6
3/6	50.8	53.3	55.7	58.3	60.7	62.9	65.2	67.4	69.6	71.7	73.9	76.0	78.0	80.0	82.0	84.1
3/7	74.5	78.0	81.3	84.6	87.8	90.9	93.9	96.9	99.8	102.6	105.5	108.2	111.1	114.0	116.8	119.7
3/8	98.0	102.1	106.2	110.2	114.1	118.0	121.8	125.6	129.3	132.9	136.6	140.3	144.1	147.8	151.5	155.1
3/9	120.5	125.5	130.4	135.2	140.0	144.6	149.3	153.9	158.4	163.0	167.6	172.2	176.8	181.4	186.0	190.5
4/5	8.8	9.2	9.6	10.0	10.3	10.7	11.1	11.5	11.8	12.1	12.5	12.8	13.1	13.4	13.7	14.0
4/6	25.1	26.2	27.2	28.0	28.8	29.7	30.5	31.3	32.0	32.8	33.9	34.9	36.1	37.1	38.3	39.4
4/7	42.7	44.4	46.2	48.5	50.7	52.6	54.6	56.6	58.5	60.3	62.2	64.1	65.9	67.6	69.3	71.0
4/8	63.8	66.9	69.8	72.8	75.6	78.4	81.1	83.7	86.3	88.8	91.3	93.8	96.2	98.7	101.0	103.4
4/9	85.8	89.6	93.3	96.9	100.4	103.9	107.3	110.7	114.0	117.3	120.5	123.8	127.0	130.1	133.3	136.4
5/6	6.7	7.1	7.4	7.8	8.1	8.4	8.8	9.1	9.4	9.6	10.0	10.3	10.5	10.8	11.1	11.3
5/7	21.0	21.9	22.8	23.5	24.2	25.0	25.7	26.4	27.0	27.7	28.5	29.2	30.0	30.8	31.9	32.8
5/8	36.9	38.4	39.9	41.7	43.3	45.1	46.7	48.4	50.1	51.8	53.5	55.1	56.7	58.3	59.8	61.3
5/9	56.1	58.6	61.1	63.6	66.2	68.6	71.1	73.4	75.8	78.1	80.3	82.5	84.7	86.9	89.1	91.2
6/7	5.1	5.5	5.8	6.1	6.4	6.8	7.1	7.3	7.6	7.9	8.1	8.4	8.7	8.9	9.1	9.4
6/8	17.9	18.7	19.5	20.2	20.8	21.5	22.1	22.7	23.3	23.9	24.6	25.2	26.0	26.6	27.4	28.1
6/9	32.3	33.7	35.1	36.6	38.1	39.7	41.2	42.6	44.0	45.4	46.7	48.2	49.6	51.0	52.4	53.7
7/8	4.0	4.3	4.7	4.9	5.2	5.5	5.8	6.1	6.3	6.5	6.8	7.0	7.3	7.5	7.7	7.9
7/9	15.4	16.2	16.9	17.5	18.1	18.7	19.3	19.9	20.3	21.0	21.5	22.2	22.8	23.4	24.1	24.7
8/9	3.1	3.4	3.7	4.0	4.3	4.5	4.8	5.1	5.3	5.5	5.7	6.0	6.2	6.4	6.6	6.8

TABLE No. 13.—Values of "Excess Load" E, which, with 2 000 Lbs. per Foot on One Rail, will give same Moments and Shears as Cooper's "E 40" Loading. Unit 1 000 lbs.

Trusses.														Plate-Girders.			
Moments.							Shears.										
$\frac{n}{N}$	Panel Lengths.						$\frac{n}{N}$	Panel Lengths.						Span.	Moment.	End shear.	Center shear.
	20	23	26	29	32	35		20	23	26	29	32	35				
.....	.....	.....	.....	.....	.....	.....	1/5..	36.7	39.9	41.4	42.0	42.4	42.6	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	1/6..	41.3	43.5	44.6	44.9	44.7	44.8	10	12.5	20.0	15.0
.....	.....	.....	.....	.....	.....	.....	1/7..	44.5	46.0	46.8	46.4	46.3	46.2	15	18.3	25.0	19.1
.....	.....	.....	.....	.....	.....	.....	1/8..	46.8	47.7	48.3	48.2	47.5	47.2	20	21.3	30.0	18.0
.....	.....	.....	.....	.....	.....	.....	1/9..	48.5	49.0	49.5	49.2	48.4	48.0	25	23.8	31.5	19.9
2/5..	31.4	35.5	38.0	38.6	37.9	38.2	2/5..	25.5	27.8	29.5	31.5	32.5	32.7	30	24.7	33.0	20.4
2/6..	37.7	38.2	36.8	37.4	37.6	38.8	2/6..	33.2	35.5	36.8	37.9	38.2	37.9	35	24.8	34.2	22.7
2/7..	37.1	38.0	38.4	38.8	38.8	39.1	2/7..	39.0	40.4	41.0	41.6	41.5	40.9	40	25.6	35.4	24.0
2/8..	38.9	39.5	39.8	39.8	39.5	39.4	2/8..	42.7	43.4	43.7	44.1	43.7	43.0	45	26.2	36.7	24.3
2/9..	40.6	40.8	40.8	40.5	40.0	39.7	2/9..	45.2	45.6	45.7	45.8	45.4	44.5	50	26.1	37.1	24.8
.....	.....	.....	.....	.....	.....	.....	3/5..	17.5	16.8	15.8	15.5	15.8	16.5	55	26.2	37.1	25.3
3/6..	33.2	36.8	38.6	38.7	37.9	36.0	3/6..	21.6	24.6	26.4	27.4	28.0	28.2	60	26.6	37.6	25.8
3/7..	38.4	38.4	39.0	39.0	38.0	36.0	3/7..	30.4	33.1	34.1	34.6	34.4	34.5	65	27.2	38.8	26.3
3/8..	37.0	38.2	39.2	39.2	38.0	36.0	3/8..	36.8	38.3	38.9	38.6	38.6	38.2	70	27.5	40.5	26.4
3/9..	37.6	38.5	39.5	39.3	38.1	35.9	3/9..	40.8	41.8	42.0	41.5	41.2	40.8	75	27.7	42.7	27.1
.....	.....	.....	.....	.....	.....	.....	4/5..	4.0	4.0	3.5	2.5	1.5	0.0	80	28.0	44.2	27.1
.....	.....	.....	.....	.....	.....	.....	4/6..	15.3	15.0	13.5	11.4	12.3	13.2	85	28.3	45.7	28.1
.....	.....	.....	.....	.....	.....	.....	4/7..	19.6	21.2	23.4	24.7	25.8	25.7	90	28.7	47.2	28.4
4/8..	38.9	38.5	36.2	33.0	30.2	27.4	4/8..	27.6	30.6	32.2	32.6	32.4	31.8	95	28.8	48.6	28.5
4/9..	39.2	38.5	36.2	32.9	30.2	27.4	4/9..	34.4	36.4	37.1	37.1	36.6	35.5	100	28.8	50.0	28.8
.....	.....	.....	.....	.....	.....	.....	5/6..	0.2	0.8	0.8	−0.4	−1.0	−2.2	105	29.8	51.1	28.7
.....	.....	.....	.....	.....	.....	.....	5/7..	13.5	13.3	12.0	10.0	9.0	9.8	110	31.3	52.0	28.6
.....	.....	.....	.....	.....	.....	.....	5/8..	18.4	19.2	20.5	22.1	23.2	23.5	115	32.7	52.9	28.7
.....	.....	.....	.....	.....	.....	.....	5/9..	26.2	28.1	30.0	30.7	30.6	30.2	120	33.8	53.7	29.0
.....	.....	.....	.....	.....	.....	.....	6/8..	11.6	11.8	10.4	8.7	8.0	7.5	125	34.9	54.4	29.5
.....	.....	.....	.....	.....	.....	.....	6/9..	16.9	17.8	19.6	20.2	20.8	21.1	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....	7/9..	9.3	9.8	8.9	7.5	6.6	6.2	.....	.....	.....	.....

Mr. Selby

practically with the curves of actual values. The equations of the straight lines are as follows:

For end shears	E, for one girder,	$=$	$25 + \frac{1}{4} L.$
" moments	"	"	"	$22 + \frac{1}{10} L.$
" center shears	"	"	"	$20 + \frac{1}{10} L.$

Shears and moments computed from values of E obtained by the foregoing formulas will differ from the actual values by very small percentages on the safe side, except for span lengths of less than 20 ft. The greatest differences are as follows: For moments—10-ft. span, 46.6%, 15-ft. span, 15.5%, 20-ft. span, 6.6%, 100-ft. span, 2.5%. For end shears—10-ft. span, 25%, 15-ft. span, 9.4%, 60-ft. span, 2.5%. For center shears—10-ft. span, 30%, 15-ft. span, 9.1%, 20-ft. span, 14.3%, 25-ft. span, 8%, 30-ft. span, 7.4%, 125-ft. span, 3.3 per cent.

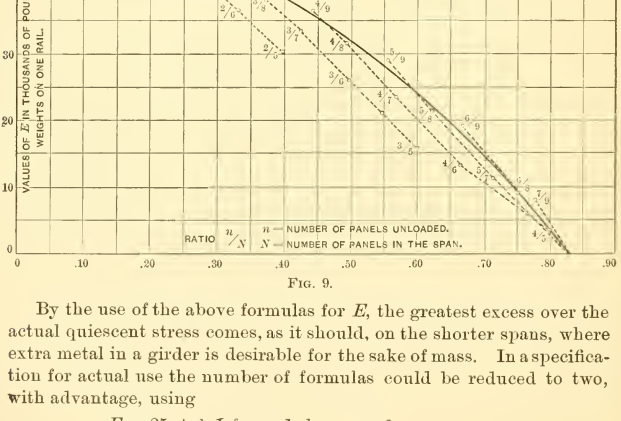


FIG. 9.

By the use of the above formulas for E, the greatest excess over the actual quiescent stress comes, as it should, on the shorter spans, where extra metal in a girder is desirable for the sake of mass. In a specification for actual use the number of formulas could be reduced to two, with advantage, using

$$E = 25 + \tfrac{1}{4} L \text{ for end shears, and}$$
$$E = 22 + \tfrac{1}{10} L \text{ for moments and center shears.}$$

The increase in values of center shears would not result in much increase in the weight of metal.

The values of E for truss shears have been plotted on the diagram, Fig. 9, taking the average value at each panel point for the different panel lengths. The abscissas of this diagram represent percentages of span unloaded. The parabola plotted will give values of E which will produce shears practically equal to the actual shears; the greatest

excesses being on the short spans and on counter shears, where they should be. The greatest percentages of difference are as follows: Mr. Selby.

$\frac{1}{5}$	20-ft. panel	7.7%
$\frac{2}{5}$	20 "	12.3%
$\frac{3}{5}$	20 "	8.4%
$\frac{3}{5}$	29 "	8.3%
$\frac{4}{6}$	20 "	3.6%
$\frac{4}{5}$	29 "	7.0%

The equation of the parabola is E, for one truss, $= 50 - 72 \frac{n^2}{N^2}$, in which $n =$ number of panels unloaded, and $N =$ whole number of panels. The values of E by this formula, for the different panel ratios, are shown in Table No. 14.

TABLE No. 14.

Panels unloaded.	Whole Number of Panels.				
	5	6	7	8	9
1	47	48	49	49	49
2	38	42	44	46	46
3	24	32	37	40	42
4	4	18	26	32	36
5	....	0	13	22	28
6	....	....	....	10	18
7	....	....	....	....	6

The use of this formula (or Table No. 14) for E, in a specification, would give shears substantially equal to, and never appreciably less than, those by Cooper's Specification.

The values of E for truss moments do not vary much below 40 for one truss. Using a constant value of 40, the greatest percentages of excess are as follows:

$\frac{2}{5}$	20-ft. panel	6.5%
$\frac{3}{6}$	20 " "	4.4%
$\frac{4}{8}$	32 " "	3.4%
$\frac{4}{8}$	35 " "	4.1%
$\frac{4}{12}$	32 " "	3.1%
$\frac{4}{9}$	35 " "	3.7%

An inspection of the diagrams, Figs. 8 and 9, shows that the method proposed by Mr. Cowles, of using the uniform train-load headed by the maximum floor-beam reaction, will generally give results too small. The panel excess derived from the floor-beam reactions for panels of 20 to 35 ft. varies from 25.6 to 27.5, and averages 26.5 for one truss.

It will be seen that the necessary excess is greater, and usually much greater, than this, except for counter shears and short plate-girders. The writer knows of one bridge now being built under

Mr. Selby. Cooper's Specifications, in which the equivalent loading is taken as 4 000 lbs. per foot, with 50 000 lbs. excess. The specifications of a prominent system call for two classes of loading: Class *A*, 5 000 lbs. per foot, with 50 000 lbs. excess; and Class *B*, 4 000 lbs. per foot, with 40 000 lbs. excess. It is evident from the diagrams that these values of 20 and 25 for E are too small to give consistently-designed bridges for a uniform train-load headed by two correspondingly heavy consolidation engines.

It may seem strange, at first glance, that the values of E should be smaller for short plate-girders than for longer ones, but it will be found that the value of E divided by the span length is greater for the short spans than for the long ones; that is, the "equivalent uniform load," increases as the span decreases.

A specification based on the foregoing formulas, viz., the use of a uniform load of 2 000 lbs. per foot for one truss or girder with a single excess, determined by—

$E = 40$ for truss moments,

$E = 50 - 72 \frac{n^2}{N^2}$ " " shears,

$E = 25 + \frac{1}{4} L$ " plate-girder end shears, and

$E = 22 + \frac{1}{10} L$ " " " moments and center shears,

would surely give a consistently proportioned structure, with stresses as nearly equivalent to the actual quiescent ones as it is possible to obtain by simple methods. The uniform load is constant, and the values of E are determined by easily memorized formulas, no tables or diagrams being necessary. Mr. Waddell's "equivalent uniform load" method is necessarily cumbersome, because the load for any given span has to be taken from a diagram plotted from computed equivalents, and cannot be memorized. The diagram must be before one when beginning the computation of a span. Mr. Waddell, in his discussion, says: "In these tables the best possible showing was made for both the 'Single-Concentration Method' and the 'Double-Concentration Method.'" It is evident that it is not the best possible showing for the single-excess method to compare results obtained by using a constant value of E with those obtained by using a variable value of uniform load computed for the cases compared. To illustrate by an extreme case, if the variable excesses from Table No. 13 were used in a comparison, the showing for the single-excess method would be very good, indeed, as the percentage of error would be zero in every case. It has been shown that the differences from using the foregoing simplified formulas are very small, except for spans less than 20 ft., and that, in all cases, the excess of metal resulting is placed where it will do the most good.

Finally, by introducing the consideration of impact, and, bearing in mind the widely varying ideas as to the proper allowances therefor, it becomes possible to simplify still further the specified loading, and

to compute at one operation both the quiescent stresses and the impact allowance. Tables Nos. 15, 16 and 17 show the percentage of excess stress from using a constant value of E for all purposes, and a comparison with the allowance for impact specified by the railroad system with which the writer is connected. The formula for impact is $I = \frac{10}{L+10}$, in which L is the span length for plate-girders and for truss moments, and is the length of span loaded for truss shears. Mr. Selby.

TABLE No. 15.—Plate-Girder Spans.

	25 ft.	50 ft.	75 ft.	100 ft.	125 ft.
Moments $E = 50$	54%	31 %	22 %	16%	9.5%
" $E = 40$	33%	18.5%	12 %	9%	3.2%
End Shears $E = 50$	33%	15 %	6.2%	0%	0 %
Impact $I = \frac{10}{L+10}$	30%	16.7%	12 %	9%	7.5%

TABLE No. 16.—Truss-Center Moments, 20-ft. Panels.

	5 panels.	6 panels.	7 panels.	8 panels.	9 panels.
$E = 50$	14%	10 %	6.5%	5.6%	5 %
Impact $I = \frac{10}{L+10}$	9%	7.7%	6.7%	6 %	5.3%

TABLE No. 17.—Truss Shears. 9 Panels at 20 ft.

Panel points	1.	2.	3.	4.	5.	6.	7.
$E = 50$	0.6%	2.3%	5.0%	10%	19%	34%	58.5%
Impact $I = \frac{10}{L+10}$	6.0%	6.7%	7.7%	9%	11%	14%	20.0%

Ideas vary so much as to the correct allowance for impact, that the above formulas will answer for comparison as well as any, and the correspondence is close enough to make it plain that the use of a constant single-excess load for all spans, except plate-girder spans under 20 ft., comes very near to giving, with one operation, the desired result, namely, the actual stress plus the impact.

The writer would, therefore, favor, for a standard railroad bridge specification, the use of a uniform load of 4 000 lbs. per foot of track, together with a single excess of 100 000 lbs., placed at the most effective point, with no other allowance for impact for all spans over about 20 ft. For spans under 20 ft. stresses would be determined by a series of the heaviest passenger-axle loads with an impact allowance of $I = \frac{10}{L+10}$.

Mr. Prichard.

HENRY S. PRICHARD, M. Am. Soc. C. E. (by letter).—If the term Railroad Bridge is understood to include the track, then the loads to which railroad bridges are subjected are concentrated wheel-loads. These loads are somewhat distributed over the remainder of the structure by the track, which slightly lessens the stresses in the connections of stringers to floor beams and reduces very considerably the pressures on ties and on local points in the flanges of stringers, but, except in these regards, the stresses produced are the same as if the loads were concentrated directly on the flanges of the stringers.

The most accurate method of specifying the loading to which a bridge is subjected is, of course, that of specifying the actual loads. For present conditions, the critical loadings are approximately knowable, therefore, for present conditions, the most accurate method of specifying the loading is to specify wheel concentrations. If, in a bridge designed for present conditions, the loads increase, but still maintain the same relations between the various wheel concentrations of both the engines and the cars, then the live-load stresses will be increased in the same proportion, and if the bridge was originally well designed for the original wheel concentrations, the distribution of the material will remain the most economical for the increased loading. Even if the relations between the wheel concentrations should vary greatly in the future, in a manner not intentionally provided for, there is no reason to suppose that any method other than that of specifying wheel concentrations would be more accurate as regards these future conditions, except by mere chance. From the foregoing facts, it is evident that, as far as accuracy is concerned, there is no reason why wheel concentrations should not continue to be specified.

Mr. Scofield and Mr. Snow have pointed out, what few if any will deny, that to those, including many bridge buyers, who are not expert in bridge design, a definite engine-loading is more intelligible than methods of supposed equivalents.

In fact, the method of specifying wheel concentrations is the only one which those who are not experts can use understandingly to give a close equivalent for the traffic conditions they anticipate, unless they obtain the assistance of an expert. Even for experts, the easiest method of specifying loads which satisfy their ideas regarding the traffic conditions, is to specify the engine wheel-concentrations.

This is shown by the fact, well illustrated in the discussion, that nearly all who have proposed methods other than by wheel-concentrations, have arrived at the loads they specify by first deciding on certain engine wheel-concentrations and then, with more or less labor (generally considerable), determining on supposedly close equivalents.

From what has been stated, it follows that, as far as facility in

writing specifications is concerned, there is no reason why the method of specifying wheel concentrations should not be continued. Mr. Prichard.

The questions of accuracy and facility of specifying the loads being decided in favor of the method of engine wheel-concentrations, the question of the wisdom of abandoning its use depends on the value of the advantage as to accuracy, compared with the increased cost, if any, by which this advantage is obtained. To those who are not familiar with the short ways which have been devised for determining the stresses from wheel concentrations, or who, from lack of practice, cannot use them with facility, the cost of making the calculations probably seems much greater than it really need be. For stringers, floor beams, hangers, plate-girders and single-intersection trusses, resting on two supports, the calculation of the stresses from concentrated wheel-loads by proficient specialists is a very simple matter; in fact, it takes less time than by using some of the other systems which have been proposed, and the cost is insignificant, as compared with the total cost of the bridge. Such cases include the great majority of railway bridge work, and for them there would seem to be little advantage in substituting for this method an approximately equivalent one, even if a satisfactory one is obtainable. Be this as it may, it is certain that before the method of wheel concentrations can be abandoned, some other method or methods must be selected to take its place. From present appearances it would be a perfectly safe proposition for the advocates of this method to agree to abandon its use when its opponents unite in recommending some other to take its place.

For the trusses of long spans it is not especially difficult to obtain other methods which give results agreeing closely with those obtained from any system of wheel concentrations of usual type, and a fair agreement can be obtained for trusses of spans of moderate length, but for short spans and for floor systems the writer knows of no other method which gives such close approximations in all cases. For multiple-intersection trusses, for continuous girders, and for some other special cases there is much more reason for adopting some approximately equivalent method, than there is in the cases previously discussed, because, for such cases, the use of the method of wheel-loads is often quite complicated, and in some instances it is extremely so. For such cases, even when wheel-loads are specified, approximately equivalent methods are frequently allowed and used.

There is one phase of the use of the method of wheel concentrations which requires notice, because misapprehension regarding it has resulted in throwing not a little discredit upon it. The fact referred to is, that this method has a superficial appearance of much nearer approach to accuracy than it really possesses, owing to the uncertainty as to future traffic conditions, the impossibility of making

Mr. Prichard. accurate allowance for the dynamic effect of the load, and other pertinent considerations. This fact has made the method of wheel concentrations a butt for ridicule. In as far as this ridicule is the means of disabusing the minds of those who would otherwise be deceived by the seeming accuracy of the method it is commendable, but to the considerable extent by which it discredits the real advantage as to the accuracy of this method, as compared with others, it is harmful. The thoughtful advocates of the use of the method of wheel concentrations advocate its use in spite of, and not in consequence of, its seeming accuracy, because, notwithstanding the fact that it is really far from accurate, it approaches nearer to accuracy than any other method. If those who believe that this advantage as to accuracy is worth what it costs in time and trouble should refrain from using this method simply because it has a superficial appearance of accuracy which is deceptive, they would be allowing their judgment to be influenced by purely sentimental considerations.

While the fact that it is not possible to obtain the stresses from the live load accurately is not a sufficient warrant for abandoning the use of the method of wheel concentrations, it affords an excellent reason for departing from the exact weights and spacings of the wheel concentrations and substituting therefor some simplified typical engine.

Mr. Wing. Charles B. Wing, Assoc. M. Am. Soc. C. E. (by letter).—With our present limited knowledge of the stresses in bridge superstructures, due to rapidly moving train loads, any refinement of calculation of stresses from assumed loads is not justified, if the cost of the structure is thereby increased. To quote an axiom familiar to bridge engineers: "Stresses should not be figured to half a pound when you guess at the load."

In discussing the advisability of discontinuing the use of the method of wheel concentrations in determining the stresses in railway bridges, certain conditions of present practice should be considered carefully. The loading now in common use is that of wheel concentrations, and there seems to be no agreement among engineers as to the best substitute for this loading.

All the substitutes generally advocated involve the use of wheel concentrations in their computation. If the method of wheel concentrations is specified by purchasers, competition among bridge manufacturers compels the use of the actual wheel concentrations specified in computing stresses. Railway bridge engineers are in the habit of testing the strength of their existing structures by actual engines. They have moment diagrams calculated for the various types of engines in use on their system, and, naturally, prefer the method of wheel concentrations for calculating stresses. Any proposed change in the present method of specifying the load must meet with the approval of railway bridge engineers if it is to be adopted for general use.

The principal objection urged against the use of the method of wheel concentrations is the greater labor involved in calculating stresses thereby. Mr. Wing.

This extra labor is so slight that its cost cannot be considered a sufficient reason for rejecting the method. Few engineers connected with bridge shops have admitted that the labor is great when a moment diagram has once been computed. Such a diagram is necessary for computing any of the substitute methods suggested.

The writer, therefore, suggests, as a solution of the question under discussion, that railway engineers, in writing specifications, include a moment diagram of the wheel concentrations as a part of the specification.

This will reduce materially the labor of the estimating department of every bridge manufactory, no matter what system of loading is finally adopted for computing stresses.

Such moment diagrams are part of the office equipment of every railway bridge engineering department, and including them in a specification involves no extra labor on the part of the bridge purchaser.

EDGAR MARBURG, M. Am. Soc. C. E. (by letter).—The current discussion of the faults and merits of the method of wheel concentrations seems to indicate that the settlement of this long-mooted question is as remote to-day as it has ever been. The so-called "equivalent-load systems" bid fair to rival, in variety and in multiplicity, the wheel systems which they are designed to supplant. The only hope of an approach to some agreement would seem to lie in a report on the subject by a Special Committee. The writer shares the view expressed by Mr. Thacher, that unification will never be achieved through the efforts of any single individual. If, on the other hand, a Committee would formulate a very limited number (perhaps not exceeding three or four) of "basic" wheel systems, in which the loads may be varied throughout by percentages chosen at pleasure, it may be reasonably expected that they would come into very general use. This excellent method of variation by percentages has already been adopted by Mr. Cooper. Mr. Marburg.

It would seem that the following general method of procedure should meet with the approval of the most ardent partisans of both factions. The opponents of wheel systems could then center their efforts upon the few standard systems which might be chosen, and derive the necessary "constants" and "variables" for the particular scheme in view, with any desired degree of elaboration. From the standpoint of the bridge companies, and expert computers generally, the objection is not so much to wheel systems as such, but to the endless and needless variety in which they appear.

In the opinion of the writer, the arguments against the method of wheel concentrations are seldom fairly presented. Without reference

Mr. Marburg. to the speculative element of increased loading in the future, there are admittedly a dozen or more important reasons in support of the well-known and universally admitted fact that the computed stresses, based on wheel systems, represent at best only fair approximations to actual conditions. There is, in fact, no adequate ground for assuming that such stresses accord any better with the maximum stresses in service than those found by any one of the "equivalent" methods, provided the two sets of computed stresses agree within reasonable limits. This latter qualification is, however, the all-important one. It may be properly claimed that no simple system of "equivalent" loading has been proposed which is sufficiently comprehensive and flexible to meet satisfactorily the many "special cases" of common occurrence in miscellaneous practice. The maximum reactions from adjacent spans of unequal lengths may be cited as a single example. This applies to viaduct posts, to end floor-beams in skew bridges, and to the suspenders for the latter. Other examples may be found in swing bridges, cantilevers, turn-tables, and unusual constructions and combinations generally. However artificial the wheel system may be, it has the great merit of consistency under all conceivable circumstances.

The comparisons between wheel systems and "equivalent" systems are usually restricted to trusses with horizontal chords, although the latter are the exception rather than the rule for spans exceeding 200 ft. While the ratio of the chord stresses in such comparisons remains, of course, unaffected, by the inclination of the chord, that of the web stresses is greatly modified. The writer has occasionally made such comparisons, but not in sufficient variety and number to warrant definite deductions.

From the standpoint of the railroad companies, the writer would not regard it expedient, or, in fact, safe, to abandon the wheel system. A prescribed engine loading should always be available for special conditions and as a general criterion in doubtful cases. Tables or diagrams could readily be appended to the specifications, giving (*a*) the maximum bending moments and shears for girders; (*b*) the shears at the center and quarter points; (*c*) the floor-beam reactions for equal panels; and (*d*) the equivalent uniform-loads for chord stresses. The latter the writer would determine from the end shear, for an assumed average panel length, rather than from the bending moment at the quarter point. The results are slightly greater, but involve no errors (differences) on the side of danger in the end panels. Compared with the chord stresses from the wheel system itself, the increase is seldom greater than 4% (usually, then, in only a single panel), and very seldom or never as high as 5 per cent. The writer has applied this method for several years in practice to a great variety of wheel loads as an approximate check on the more refined methods, with results as stated.

Mr. Marburg.

The writer prefers this method of equivalent loads also for web stresses in trusses with parallel chords. It obviates the differences on the side of danger incidental to the quarter-moment method,* provided the forward half-panel loads be neglected, as should be done in all cases.

For the web stresses in trusses with inclined chords, the writer is not prepared to endorse any of the so-called equivalent methods, nor is he aware that this phase of the subject has ever been sufficiently investigated.

The writer prefers the above-mentioned equivalent uniform-load method to the methods of added single and double concentrations, not because it accords better with the stresses from wheel loads (in fact, the contrary is true), but on account of its greater simplicity. Besides, where its use is restricted as outlined above, the errors are all on the side of safety.

The possibilities of the added-concentration methods in the way of so-called accuracy cannot be revealed adequately by the method chosen by Mr. Waddell in compiling his tables of comparative values. Either the concentrated load should be constant and the uniform load variable, as suggested some years ago,† or *vice versa*, as here proposed by Mr. Selby.

* These differences may be much greater than those exhibited in Mr. Waddell's tables in the present discussion. See Johnson's "Modern Framed Structures," pp. 107, 108.

† *Ibid*, p. 102.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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No. 859.

GENERAL CRITERION FOR POSITION OF LOADS CAUSING MAXIMUM STRESS IN ANY MEMBER OF A BRIDGE TRUSS.

BY L. M. HOSKINS, ESQ.

PRESENTED APRIL 5TH, 1899.

WITH DISCUSSION.

The most dangerous load to which a railway bridge is subjected consists of a somewhat irregularly distributed series of train weights, applied to the track at the points of contact of the wheels, and to the trusses at the points of support of the track system. The actual distribution of such a load among the joints of the trusses is in some degree uncertain; some assumption must be made before the problem of computing stresses becomes determinate. In view of this uncertainty, some engineers prefer to simplify the problem by replacing the actual load by an "equivalent uniform-load," or by a series of equal panel loads, or by some combination of these. Others believe that a closer approximation to actual conditions is obtained by using a series of concentrated loads representing as nearly as possible the actual weights carried by the wheels of the locomotives and cars, and assuming each load to be applied to the truss as if carried by a simple beam supported at the two adjacent panel joints.

It is no part of the object of this paper to consider the relative merits of these different assumptions. The results of the following discussion, although especially useful in treating the case of actual wheel loads, are wholly general, so far as the character of the load series is concerned. This paper is offered as a contribution toward the unification of the theory of maximum stresses due to moving loads.

This theory, with applications to the various cases of practical importance, is quite fully treated in manuals and text-books, and in special papers dealing with particular methods.* The problem presents no serious difficulties of principle, but the treatment usually given lacks generality. This is especially true of that part of the theory which deals with the criterion for position of loads causing maximum stress, a special analysis being commonly given for each of a number of particular cases. From this lack of generality arises the chief difficulty in applying the rules deduced. The use of a special rule or formula in each of a number of cases, each involving a special notation, is likely to prove confusing, even when each case in itself is sufficiently simple.

In the endeavor to generalize the treatment of this problem, the author was led to a result simple in form and easily applicable to the various particular cases. By the use of a simple general notation, the criterion for maximum stress assumes the same form for all members, whatever their position in the truss.† The deduction of this general criterion, with examples illustrating its application in various cases, is the object of this paper.

Generality of the Discussion.

It will be well to make clear at the outset the exact degree of generality of the discussion.

As regards form of truss and choice of member, there is no restric-

* "The Stresses in Framed Structures," by A. Jay Du Bois.
"The Theory and Practice of Modern Framed Structures," by J. B. Johnson, C. W. Bryan and F. E. Turneaure.
"Text-Book on Roofs and Bridges," by Mansfield Merriman and H. S. Jacoby.
"On the Calculation of the Stresses in Bridges for the Actual Concentrated Loads," by George F. Swain. *Transactions*, Am. Soc. C. E., Vol. xvii (1887).
"A New Graphical Solution of the Problem, What Position a Train of Concentrated Loads Must Have in Order to Cause the Greatest Stress in Any Given Part of a Bridge Truss or Girder," by Henry T. Eddy. *Transactions*, Am. Soc. C. E., Vol. xxii (1890).

† The general formula expressing this criterion was given by the author in a paper presented to the Wisconsin Academy of Sciences, Arts and Letters, in December, 1893, and published in Vol. x of the *Transactions* of the Academy. The proof there given, though rigorous, was less simple than that now offered. A third proof, based upon a general graphical treatment of the problem, and depending upon elementary properties of the funicular polygon, is given in the author's "Elements of Graphic Statics," Revised Edition (1899).

tion, except that the stress must be "simply determinate." By this it is to be understood that the member under discussion may be taken as one of three, through which a section may be passed completely dividing the truss. Figs. 1 to 6 show six cases to which the analysis applies, the member under consideration being in every case marked k, and the section dividing the truss being represented by a broken line.

As regards the load series, this may consist of any distributed or concentrated loads. A concentrated load, however, is to be regarded as distributed over a finite, though short, horizontal length. This does not limit the discussion, so far as practical applicability is concerned, since the concentrated loads of practice do, in fact, act in the manner assumed. A concentrated load, in the strict mathematical sense (a finite load applied at a mathematical point), has only an ideal existence. Even such ideal loads may be regarded as limiting cases of distributed loads, and there is no difficulty in giving a discussion rigorously applicable to such an ideal load series. But the language used in the following analysis is exact only when such ideal concentrated loads are excluded.

Definitions and Notation.

Stress-Moment and Moment-Center.—Suppose the stress in any member to be determined by the "method of moments." For this purpose the truss is conceived to be divided by a section cutting the member in question and two others; the intersection of the axial lines of these two is taken as the origin of moments in applying the equation of equilibrium to either of the two parts of the truss. Let this origin be called the moment-center for the member under consideration, and let the moment of the required stress, with respect to this origin, be called the stress-moment. Numerically, the value of the stress-moment is equal to the sum of the moments of the external forces acting upon either of the two portions into which the truss is divided by the section; algebraically, two cases are to be distinguished by the signs plus and minus. Calling clock-wise rotation negative, let the stress-moment be called positive when it resists a tendency of the left-hand portion of the truss to rotate negatively, and of the right-hand portion to rotate positively, with reference to the moment-center.

The kind of stress in any member may be determined by inspection

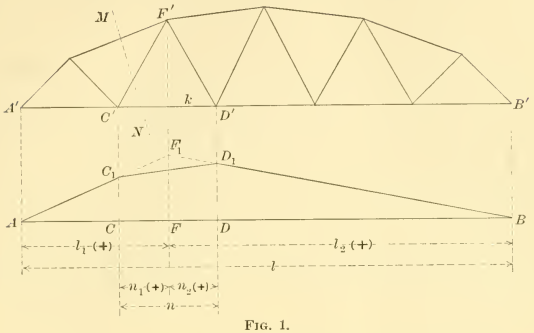

FIG. 1.

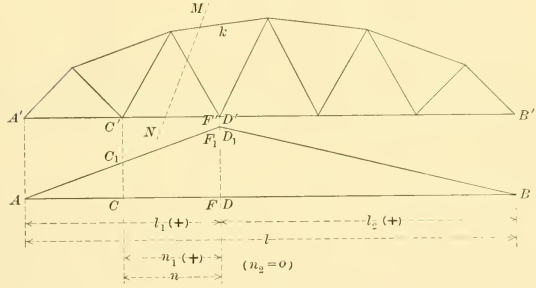

FIG. 2.

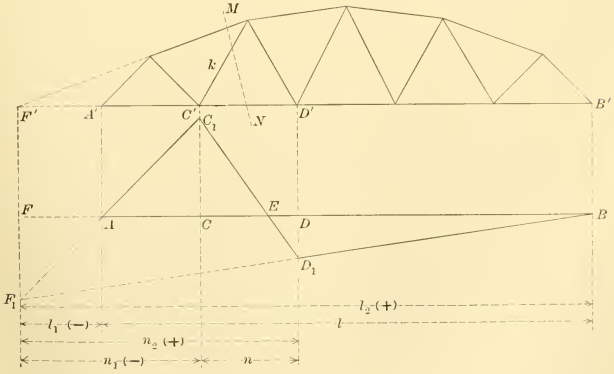

FIG. 3.

of the truss diagram, when the sign of the stress-moment and the position of the moment-center are known.

Notation.—Of the three members cut, in dividing the truss, one belongs to the chord at whose joints the moving loads are applied to the truss. Let the ends of this member be designated in all cases by C' and D', and let A' and B' be the points of support of the truss. Let the moment-center for any chosen member be designated by F'. Projecting the five points A', B', C', D' and F' upon a horizontal line, let their projections be marked A, B, C, D and F. The relative positions of these points in each of several cases are shown in Figs. 1 to 6, the member whose stress is under consideration being in every case marked k.

$$\text{Let } A\,B = l,\ A\,F = l_1,\ F\,B = l_2;$$
$$C\,D = n,\ C\,F = n_1,\ F\,D = n_2.$$

Let the signs of the four quantities l_1, l_2, n_1 and n_2, conform to the order of the letters used in defining them, the positive direction being from left toward right. Thus l_1 and l_2 are both positive if F falls between A and B; n_1 and n_2 are both positive if F falls between C and D. In all cases $l_1 + l_2 = l$ and $n_1 + n_2 = n$.

Let the load per unit length at any point be denoted by w. The only restriction as to the value of w is that it shall be either finite or zero at every point. The only case thus excluded is that of ideally concentrated loads above mentioned. At the point of application of an actual concentrated load w becomes very great, but not infinite.

Let the values of w at A, B, C and D, respectively, be denoted by w_a, w_b, w_c and w_d.

Let W = total load on the span $A\,B$;
Q = " " " panel $C\,D$;
P_1 = " " $A\,C$;
P_2 = " " $D\,B$.
Identically, $W = P_1 + P_2 + Q$.

Deduction of Criterion for Maximum or Minimum Stress.

In the following analysis, direct reference will be made to the case shown in Fig. 1, in which the four quantities l_1, l_2, n_1 and n_2 are all positive. It will be seen, however, that the reasoning applies to any of the cases shown in Figs. 2 to 6 if regard be had for algebraic signs.

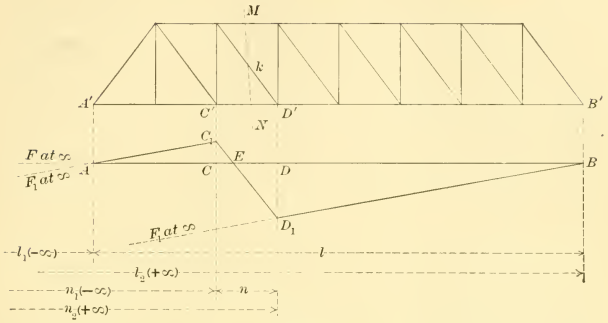

FIG. 4.

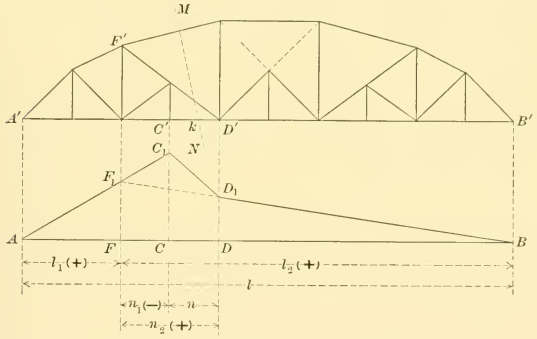

FIG. 5.

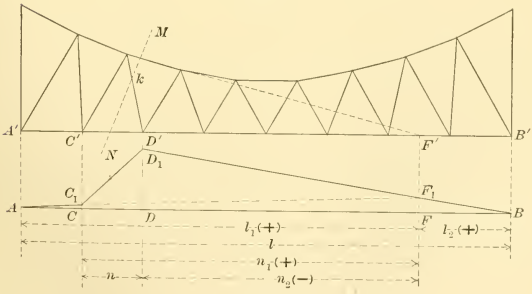

FIG. 6.

Value of Stress-Moment.—The magnitude of the resultant load on AC is P_1; let the distance of its line of action from A be p_1. Similarly, let P_2, the resultant of the loads on DB, act in a line distant p_2 from B; and let Q, the resultant of the loads on CD, act in a line distant q_1 from C and q_2 from D.

The stress-moment due to P_1 may be found by applying the principles of equilibrium to the portion of the truss at the right of the section MN (Fig. 1). The reaction at B is $\frac{P_1 p_1}{l}$; the moment of this reaction with respect to the moment-center is $\frac{l_2}{l}(P_1 p_1)$, which is therefore the value of the stress-moment due to P_1.

The stress-moment due to P_2 is found in a similar manner, by considering the equilibrium of the portion of the truss to the left of the section MN; the value being $\frac{l_1}{l}(P_2 p_2)$.

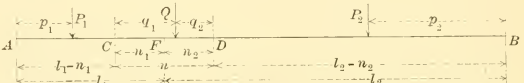

FIG. 7.

Of the load Q, the portion $\frac{Qq_2}{n}$ is applied to the truss at C, and the portion $\frac{Qq_1}{n}$ at D. The effect of the former is found as in the case of P_1; $l_1 - n_1$ taking the place of p_1 and $\frac{Qq_2}{n}$, the place of P_1; the stress-moment due to this part of Q is therefore $\frac{(l_1 - n_1)\, l_2}{nl}\ (Qq_2)$; while the stress-moment due to the portion at D, determined as in the case of P_2, is in like manner found to have the value $\frac{(l_2 - n_2)\, l_1}{n\, l}\ (Qq_1)$. These values are all positive in the case shown in Fig. 1.

The stress-moment due to the total load on the truss is the sum of these four quantities, or

$$M = \frac{l_2}{l}\left(P_1\, p_1\right) + \frac{l_1}{l}\left(P_2\, p_2\right) + \frac{(l_2 - n_2) l_1}{n\, l}\left(Q\, q_1\right) + \frac{(l_1 - n_1) l_2}{n\, l}\left(Q\, q_2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

Change of Stress-Moment Due to Small Displacement of Loads.—Let the load-series receive a small displacement δx toward the left (this being taken as the positive direction for displacements), and let δM denote the resulting increment in the value of M. To compute δM it is necessary to determine the increments of the four quantities $P_1 p_1$, $P_2 p_2$, $Q q_1$ and $Q q_2$. Of these quantities, the first is the moment of P_1 with respect to A, the second is the moment of P_2 with respect to B, the third is the moment of Q with respect to C, and the fourth is the moment of Q with respect to D.

During the displacement, loads $w_a \delta x$, $w_b \delta x$, $w_c \delta x$ and $w_d \delta x$ pass the points A, B, C and D, respectively.

After the displacement, the load which has replaced P_1 is the resultant of three components, as follows: (1) The original load P_1, acting in a line distant $p_1 - \delta x$ from A, (2) a load $w_c \delta x$, acting just at the left of C, this being the amount of load which passes C during the displacement, and (3) a negative load equal to $w_a \delta x$, acting just at the left of A, this being the amount of the load which passes off the truss at A during the displacement. The new value of $P_1 p_1$ is the sum of the moments of these three components with respect to A. Therefore, neglecting powers of δx higher than the first, we have, as the value of $P_1 p_1$, after the displacement, $P_1 (p_1 - \delta x) + w_c (l_1 - n_1) \delta x$.

Similarly, the load which has replaced P_2 is the resultant of a load P_2 acting at a distance $p_2 + \delta x$ from B, a load $w_b \delta x$ acting just at the left of B, and a negative load $w_d \delta x$ acting just at the left of D. The new value of $P_2 p_2$ is the sum of the moments of these three components about D, or $P_2 (p_2 + \delta x) - w_d (l_2 - n_2) \delta x$.

Similar reasoning gives for the new values of $Q q_1$ and $Q q_2$, $Q (q_1 - \delta x) + w_d n \delta x$ and $Q (q_2 + \delta x) - w_c n \delta x$.

The value of M after the displacement is, therefore,

$$M + \delta M = \frac{l_2}{l}\left[P_1 (p_1 - \delta x)\right] + \frac{l_1}{l}\left[P_2 (p_2 + \delta x)\right]$$
$$+ \frac{(l_2 - n_2)\, l_1}{n\, l}\left[Q (q_1 - \delta x)\right] + \frac{(l_1 - n_1)\, l_2}{n\, l}\left[Q (q_2 + \delta x)\right]$$

the terms involving w_c and w_d disappearing from the simplified result.

The increment of M caused by the displacement is

$$\delta M = \left(-\frac{l_2}{l} P_1 + \frac{l_1}{l} P_2 + \frac{l_1 n_2 - l_2 n_1}{l\, n} Q\right) \delta x.$$

Dividing by δx and passing to the limit, δx approaching zero,

$$\frac{d\,M}{d\,x} = \frac{l_1}{l}\left(P_2 + \frac{n_2}{n}\,Q\right) - \frac{l_2}{l}\left(P_1 + \frac{n_1}{n}\,Q\right) \ldots\ldots\ldots\ldots (2)$$

Condition for Maximum or Minimum Stress.—The condition for maximum or minimum value of M (and therefore also of the stress), is $\frac{d\,M}{d\,x} = 0$; which gives the equation

$$\frac{P_1 + \frac{n_1}{n}\,Q}{l_1} = \frac{P_2 + \frac{n_2}{n}\,Q}{l_2} = \frac{W}{l} \ldots\ldots\ldots\ldots\ldots (3)$$

the third member of the equation being obtained by adding the numerators and denominators, respectively, of the fractions forming the first and second members.

This general equation is satisfied whenever the stress in the member is a maximum or a minimum. That it applies to any one of the cases shown in Figs. 2, 3, 4, 5 and 6, may be verified by following the above reasoning through, taking account of the algebraic sign of every quantity dealt with.*

Special Forms of the General Criterion.—For practical use it will generally be found convenient to put Equation (3) in one of the following forms:

$$P_1 + \frac{n_1}{n}\,Q = \frac{l_1}{l}\,W \ldots\ldots\ldots\ldots\ldots\ldots (3a)$$

$$P_2 + \frac{n_2}{n}\,Q = \frac{l_2}{l}\,W \ldots\ldots\ldots\ldots\ldots\ldots (3b)$$

unless the moment-center is at infinity. In this case (Fig. 4) these equations take indeterminate forms. But since when this occurs $\frac{l_1}{n_1} = 1$ and $\frac{l_2}{n_2} = 1$, both equations may be reduced to the form:†

$$Q = \frac{n}{l}\,W \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3c)$$

In addition to the general criterion expressed by Equation (3), it is necessary to have some means of distinguishing between positions giving maximum and those giving minimum values of the stress. Further, since the stress in any member will, in general, pass through a number

* For the case represented in Fig. 1, in which the member under consideration belongs to the loaded chord, l_1, l_2, n_1 and n_2 being all positive, formulas identical with Equation (3) have been given by several writers. The generality of the formula seems not to have been recognized.

† Equation (3c) is identical with the condition for maximum shear in the panel containing the web member under consideration. It is, of course, only when the chords are parallel that the maximum shear and maximum stress can be assumed to occur simultaneously.

of maximum values as the loads pass over the bridge, means are needed for selecting the greatest of the possible maximum values.

A general criterion for distinguishing between positions giving maximum and those giving minimum values will be deduced below. It will be well, however, first to consider the question in a somewhat less general manner. The problem of distinguishing between maxima and minima, as well as that of selecting the greatest from the possible maximum values, may be treated advantageously by the use of "influence diagrams."

Influence Diagrams.

A very full and clear discussion of influence lines, with applications to the determination of maximum stresses in various cases of practical importance, has been given by George F. Swain, M. Am. Soc. C. E., in the paper cited on page 241. It will be advantageous to present here a brief treatment of this subject from the general point of view adopted in the foregoing analysis.

Influence Line.—The influence line, for a given member of a truss, is a line whose ordinate at any point represents the value of the stress in the member due to a unit load at that point.

General Method of Drawing Influence Line.— In the following discussion any ordinate of the influence line will be regarded as agreeing in sign with the corresponding value of the stress-moment, according to the convention already explained. Positive ordinates will be drawn upward from the reference line, negative ordinates downward.

If M denotes the stress-moment due to a unit load in a given position, and t the perpendicular distance from the moment-center to the axial line of the member under consideration, $\frac{M}{t}$ denotes the value of the ordinate of the influence line.

Referring first to the case in which l_1, l_2, u_1 and u_2 are all positive (Fig. 1), and repeating the reasoning already given in determining values of the stress-moment due to P_1, P_2 and Q, the following results are obtained:

A unit load between A and C, distant x from C, causes a stress $\frac{l_2 x}{lt}$. This changes from 0 to $\frac{l_2 (l_1 - u_1)}{lt}$ as the load passes from A to C.

Similarly, as a unit load passes from B to D, the stress due to it changes from 0 to $\frac{l_1 (l_2 - n_2)}{l t}$.

Of a unit load on $C D$, distant x from C, the portion $\frac{n - x}{n}$ comes upon the truss at C and $\frac{x}{n}$ at D. The former causes the stress $\frac{l_2 (l_1 - n_1)}{l t} \times \frac{n - x}{n}$, and the latter the stress $\frac{l_1 (l_2 - n_2)}{l t} \times \frac{x}{n}$. As the load passes from C to D, the sum of these values changes from $\frac{l_2 (l_1 - n_1)}{l t}$ to $\frac{l_1 (l_2 - n_2)}{l t}$.

From these results it is seen that the influence line consists of three straight lines $A C_1$, $C_1 D_1$, $D_1 B$; the ordinates, $C C_1$ and $D D_1$, having values:

$$C C_1 = \frac{l_2 (l_1 - n_1)}{l t}; \quad D D_1 = \frac{l_1 (l_2 - n_2)}{l t}.$$

If the various cases shown in Figs. 1 to 6 be considered, these values are found to be correct in magnitude and sign for all cases. The following general property may also readily be verified:

The lines $A C_1$ and $B D_1$ (produced if necessary) have a common ordinate at the point F, its value being $\frac{l_1 l_2}{l t}$.

This property gives the following general rule for drawing the influence line:

At F erect an ordinate equal in magnitude and sign to $\frac{l_1 l_2}{l t}$, and draw straight lines from its extremity to the points A and B. Connecting C_1 and D_1, the points in which these lines intersect ordinates erected at C and D, respectively, the line $A C_1 D_1 B$ is the influence line.

It will be seen that, so long as the moment-center falls within the span, so that both l_1 and l_2 are positive, the ordinates of the influence line are everywhere positive. This is the case in Figs. 1, 2, 5 and 6. There are, however, two types of influence line with ordinates everywhere positive, corresponding to the cases in which F falls within the panel CD (Fig. 1), and without the panel (Figs. 5 and 6). The limiting case between these two is shown in Fig. 2.

If the moment-center falls without the span, making l_1 or l_2 negative, the ordinates of the influence line are positive in one portion of

the span and negative in the remaining portion. Fig. 3 represents the case in which l_1 is negative, the ordinate CC_1 being positive and DD_1 negative. The line $C_1 D_1$ crosses $A B$ at a point E, between C and D.

If the point F falls at infinity, as in Fig. 4, the above general rule for drawing the influence line by means of the ordinate at F becomes inapplicable. The lines $F_1 A$, $F_1 B$ are in this case parallel, and may be drawn by means of the general values previously given for the ordinates at C and D. It may be noticed also that if $A C_1$ be prolonged, its ordinate at B has the value $-\frac{l_2}{t}$; and if $B D_1$ be prolonged, its ordinate at A has the value $-\frac{l_1}{t}$. These values are determinate even when l_1, l_2 and t are infinite.

When the load series consists of concentrated loads, the question whether any certain position, in which the general equation (3) is satisfied, gives a maximum or a minimum value of the stress may usually be determined by simple inspection of the influence diagram. It is also often possible to determine, with high probability, which of several maximum values is greatest, without actually determining the values. These questions are now to be considered.

Discrimination Between Maximum and Minimum Values.

Application of Influence Diagram.—Referring to Fig. 1, and observing the slope of each of the three straight lines forming the influence diagram, consider how the stress in the member k, due to a single load, varies as the load moves uniformly from B toward the left. During the motion from B to D the stress increases at a uniform rate; from D to C it decreases at a uniform rate; from C to A it decreases uniformly at a greater rate. If the load moves from A to B, these effects are reversed.

Next consider the effect of displacing a series of loads. As above, let the total loads on $A C$, $C D$ and $D B$, respectively, be denoted by P_1, Q and P_2. If the loads are displaced toward the left, the stress due to P_2 increases, while the stress due to P_1, and also that due to Q, decreases. The resultant of these three effects may be either an increase or a decrease; but the resultant rate of change of the stress (whether increase or decrease) remains constant as long as P_1, P_2 and Q remain constant, that is, as long as no load passes any one of the four points A, C, D and B.

As the loads approach a position causing a maximum stress, the value of the stress increases; as they recede from such a position, the value of the stress decreases. The reverse changes take place as the loads approach and recede from a position of minimum stress.

First, suppose the stress is increasing as the loads move toward the left. If a load comes on the truss at B, the rate of increase of the stress will become greater. The same will be true if a load passes off at A. If a load passes C or D, the rate of increase of the stress becomes less, and this effect may be sufficiently great to change the increase into a decrease; if so, the stress passes through a maximum value. Hence, only when a load is at C or at D can the stress be a maximum.

Next, suppose the stress is decreasing as the loads move toward the left. If a load passes C or D, the rate of decrease becomes greater. If a load comes on the truss at B, or if one leaves the truss at A, the rate of decrease becomes less, and this effect may be so great as to change the decrease into an increase; if so, the stress passes through a minimum value. Therefore, only when a load is at A or at B can the stress be a minimum.

Like reasoning obviously applies to the cases represented in Figs. 2, 3, 4, 5 and 6.*

Thus, in Fig. 2, a maximum can occur only with a load at D. A load at C can give neither a maximum nor a minimum, since $A\,C_1$ and $C_1\,D_1$ form a continuous straight line.

In the case of a web member, represented in Fig. 3, the stress may be either tension or compression. The kind of stress corresponding to a positive stress-moment (tension in this case), can have a maximum value with a load at C or at B, a minimum value with a load at A or at D. For the opposite kind of stress, these statements must be reversed. Like statements hold for the case in which F is at infinity (Fig. 4).

In Fig. 5, a maximum value of the stress in the member k can occur only when a load is at C; a minimum value may occur when a load is at A, at D or at B.

In Fig. 6, a maximum value can occur with a load at D, a minimum value with a load at A, at C or at B. This form of truss, though not of practical importance, is shown in order to illustrate the generality of the method. Its similarity to the case shown in Fig. 5 is apparent from

*The actual value of the rate of increase of the stress as the loads move may be computed from the slope of the three portions of the influence line. Using the values of the ordinates at C and D, as above given, this method leads easily to Equation (2), and thus also to the general criterion (3).

a comparison of the influence diagrams. It may be remarked that, as a special case of Fig. 6, the point F may coincide with B (or with A), making l_2 (or l_1) zero. The ordinate $F F_1$ would be zero, and the influence line would have to be constructed from the values of the ordinates at C and D, one of which would be zero.

Critical Points.—It appears from the foregoing discussion that, when the load-series consists wholly of concentrated loads, the stress can have a maximum or a minimum value only when a load is passing some one of the four points A, B, C and D. This might be inferred also from Equation (3), since only in exceptional cases can this equation be satisfied without there being a load at one of these points. Such a load may, by a slight displacement, be divided in any desired ratio between the adjacent segments of the span.

These four points may be called critical points, and may be distinguished as "maximum points" and "minimum points," according as the passage of a load through them is associated with a maximum or a minimum value of the stress.

The foregoing discussion shows that, at a maximum point, the influence line is convex upward (*i. e.*, toward the positive side), while at a minimum point it is concave upward. In applying this statement to the points A and B, the influence line is to be extended beyond the span as a horizontal line.

When only a single one of the four critical points is occupied by a load, or if such occupied points are either all maximum points or all minimum points, the form of the influence line thus shows at a glance whether the stress is a maximum or a minimum. To make the discussion complete it is necessary to consider cases in which Equation (3) is satisfied while loads are passing both maximum and minimum points.

General Criterion.—In order to deduce a completely general rule for discriminating between maximum and minimum values, consider Equation (2) deduced above:

$$\frac{d\,M}{d\,x} = \frac{l_1}{l}\left(P_2 + \frac{n_2}{n}\,Q\right) - \frac{l_2}{l}\left(P_1 + \frac{n_1}{n}\,Q\right) \ldots\ldots\ldots\ldots (2)$$

This value of $\frac{d\,M}{d\,x}$ remains constant, except when the load (either concentrated or distributed), passes some one or more of the "critical" points A, B, C or D, thus changing the values of one or more of the quantities P_1, P_2 or Q.

Assuming a small displacement of the load-series, let loads W_a, W_b, W_c and W_d, respectively, pass the four critical points A, B, C and D. The change in the value of $\frac{dM}{dx}$ may be found from the changes in P_1, P_2 and Q. For a positive displacement (the loads moving toward the left),

P_1 changes by the amount $W_c - W_a$;
P_2 " " $W_b - W_d$;
Q " " $W_d - W_c$.

Hence the change (written as an increase) in the value of $\frac{dM}{dx}$ is

$$\delta\left(\frac{dM}{dx}\right) = \frac{l_1}{l}\left[W_b - W_a + \frac{n_2}{n}(W_a - W_c)\right] - \frac{l_2}{l}\left[W_c - W_a + \frac{n_1}{n}(W_a - W_c)\right];$$

which reduces to

$$\delta\left(\frac{dM}{dx}\right) = \frac{l_2}{l}W_a + \frac{l_1}{l}W_b - \frac{n_2}{n}W_c - \frac{n_1}{n}W_d \quad \ldots\ldots\ (4)$$

According as this value is positive or negative, the value of $\frac{dM}{dx}$ increases or decreases as the loads pass through the supposed position.

As a position of maximum stress is passed, $\frac{dM}{dx}$ decreases; as a position of minimum stress is passed, $\frac{dM}{dx}$ increases.

This is the general criterion for discriminating between maximum and minimum values of the stress. The application to special cases, in which l_1, l_2, n_1 and n_2 are known in magnitude and sign, presents no difficulty.

The conclusions above reached from the study of the influence diagrams for the six cases, may easily be verified by inspection of Equation (4). Thus, if l_1, l_2, n_1 and n_2 are all positive (Fig. 1), the first and second terms in the value of $\delta\left(\frac{dM}{dx}\right)$ are intrinsically positive, while the third and fourth are negative. Therefore, for a maximum stress either W_c or W_d must be present, while for a minimum stress either W_a or W_b must be present. In the case shown in Fig. 2, the third term in the value of $\delta\left(\frac{dM}{dx}\right)$ drops out, n_2 being zero; the conclusions are otherwise the same as in the preceding case.

In Fig. 3, l_1 and n_1 being negative, the "A" and "D" terms in the value of $\delta\left(\frac{dM}{dx}\right)$ are intrinsically positive, while the "B" and "C"

terms are negative. Hence A and D are minimum points, while B and C are maximum points. The same conclusions apply to the case shown in Fig. 4, although every term in the second member of Equation (4) is infinite.

In Fig. 5, n_1 is negative, while l_1, l_2 and n_2 are positive. Therefore, in Equation (4) the "A," "B" and "D" terms are positive, while the "C" term is negative. Hence C is the only maximum point.

In Fig. 6, n_2 being negative, the only negative term in Equation (4) is the "D" term. Hence D is the only maximum point.

If all but one of the four terms in the value of $\delta\left(\frac{d\,M}{d\,x}\right)$ are zero when the condition for maximum or minimum stress (Equation 3) is satisfied, the discrimination between maximum and minimum values is thus very simple, being readily accomplished, either by simple inspection of the influence diagram, or by noticing the sign of the single term in the second member of Equation (4). If two or more terms remain, Equation (4) must be applied.

Distributed Loads.—Consider next the case in which there is a distributed load at each of the four points A, B, C and D. Equation (4) is still applicable, but its form may conveniently be changed. In accordance with the notation used in the deduction of Equation (1), let w denote the load per unit length at any point, and let w_a, w_b, w_c and w_d be the values of w at A, B, C and D. If the loads receive a positive displacement δx, we must put, in Equation (4):

$$W_a = w_a\,\delta x,\ W_b = w_b\,\delta x,\ W_c = w_c\,\delta x \text{ and } W_d = w_d\,\delta x.$$

The result is:

$$\delta\left(\frac{d\,M}{d\,x}\right) = \left(\frac{l_2}{l}\,w_a + \frac{l_1}{l}\,w_b - \frac{n_2}{n}\,w_c - \frac{n_1}{n}\,w_d\right)\delta x \quad (4a)$$

or

$$\frac{d^2\,M}{d\,x^2} = \frac{l_2}{l}\,w_a + \frac{l_1}{l}\,w_b - \frac{n_2}{n}\,w_c - \frac{n_1}{n}\,w_d \quad (5)$$

A position of the loads satisfying Equation (3) gives a maximum or a minimum value of the stress, according as the value of $\frac{d^2\,M}{d\,x^2}$ is negative or positive. If the value of w is known at every point, the application of Equation (5), to any particular case in which l_1, l_2, n_1 and n_2 are known, presents no difficulty.

Combination of Concentrated and Distributed Loads.—Another case is that in which there are concentrated loads at certain of the critical

points, while at others the load is distributed. This occurs when the load consists of one or more locomotives followed by a uniformly distributed train-load, as often prescribed in bridge specifications. The value of $\delta \left(\frac{dM}{dx}\right)$ will then contain part of the terms in Equation (4) and part of those in Equation (4*a*). Since the latter terms contain the factor δx, they will be very small in comparison with the former, and may, therefore, generally be disregarded in applying the equation.

Determination of Greatest Maximum.

In those cases in which loads in all positions cause the same kind of stress in the member under consideration, it is evident that for greatest stress the span should be loaded as fully as possible, and that the heaviest loads should be so placed as to produce as great an effect as possible. Thus in Figs. 1, 2 and 6 the heavy loads should be near *D;* in Fig. 5, near *C*. This is, of course, only a rough, general rule.

When a reversal of stress is possible, as in Figs. 3 and 4, the two kinds of stress must be considered separately. Thus, for one kind of stress, loads should cover only the portion *A E*; for the other kind, only the portion *E B*. These rules are not absolute; it may be, that, if loads are carried beyond *E*, the diminution of stress due to such loads will be more than counterbalanced by an increase due to bringing other loads nearer to their positions of greatest effect.

By inspection of the influence diagram it is thus possible to determine, with great probability in many cases, which of two maximum values is greater, without actually computing the values. In some cases, however, it may be necessary to compute the stress for each of two or more positions, in order to determine with certainty which maximum value is the greatest.

General Results.

The determination of the position of a series of moving loads which will cause the greatest stress in any member of a truss thus involves the following steps:

(1) The approximate determination of the required position by inspection of the influence diagram.

(2) The testing of the exact criterion expressed by Equation (3), or one of the equivalent Equations (3*a*), (3*b*), (3*c*).

(3) The application of Equation (4) or Equation (5) to determine with certainty whether the stress is a maximum or a minimum. In the majority of practical cases this is, however, unnecessary, the inspection of the influence diagram being sufficient.

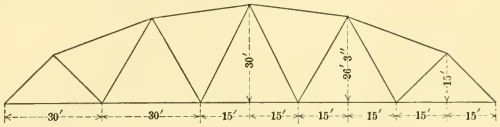

FIG. 8.

NUMERICAL APPLICATIONS.

In order to illustrate clearly the application of the general criterion to different cases, numerical examples will now be solved, involving the cases shown in Figs. 1, 3, 4 and 5.

The load-series will be assumed to consist of two locomotives followed by a uniformly distributed load, the loads and intervals being given in Fig. 9. In practice, a diagram should be drawn, showing the lines of action properly spaced, and a skeleton diagram of the truss

10	35	60	85	110	125	140	155	170	180	205	230	255	280	295	310	325	340	*Total Load* (*thou's of lbs.*)
10	25	25	25	25	15	15	15	15	10	25	25	25	25	15	15	15	15	*Loads* (*thou's of lbs.*)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	*2.5 per foot*
	7.5′	5′	5′	5′	9′	5′	6′	5′	8′	7.5′	5′	5′	5′	9′	5′	6′	5′	5′	
	7.5′	12.5′	17.5′	22.5′	31.5′	36.5′	42.5′	47.5′	55.5′	63.0′	68.0′	73.0′	78.0′	87.0′	92.0′	98.0′	103.0′	*Dist. from 1st. Load*

FIG. 9.

prepared on a movable strip of paper or on tracing cloth, to facilitate the trial of different positions.

(1) *Member of Loaded Chord.*—Consider, first, the case shown in Fig. 1, the dimensions of the truss being as marked on Fig. 8. For greatest stress in the member k the truss must be fully loaded, and the effect of the heaviest loads will be greatest if these are near the point D. A maximum value can occur only with a load at C or at D. It remains to test the general equation (3) for such positions as these general considerations suggest.

From the dimensions of the truss, we have $l = 150$ ft., $l_1 = 45$ ft.,

$l_2 = 105$ ft., $n = 30$ ft., $n_1 = 15$ ft. and $n_2 = 15$ ft. Equation $(3a)$ therefore becomes $P_1 + \frac{1}{2} Q = \frac{3}{10} W$.

It is seen that there is a conflict between the requirements that the span shall be fully loaded, and that the heaviest loads shall be near D.

It is found that there are two positions in which the conditions for maximum stress are satisfied—with either the fourth or the fifth load at C. Between two maxima there must be a minimum; and it is found that between the two positions of maximum stress there is another position in which Equation $(3a)$ is satisfied. The numerical data for testing these three positions and two others are shown in Table No. 1.

Table No. 1.

Position.	W	$\frac{l_1}{l} W$	P_1	Q	$P_1 + \frac{n_1}{n} Q$	
Third load at C..	401 250	120 375	35 000 60 000	105 000 120 000 80 000 95 000	87 500 95 000 100 000 107 500	Criterion not satisfied.
Fourth load at C	413 750	124 125	60 000 85 000	95 000 110 000 70 000 85 000	107 500 115 000 120 000 127 500	Criterion satisfied. Maximum.
Fourth load 4.5 ft. left of C....	425 000	127 500	85 000	85 000	127 000	Criterion satisfied. Minimum.
Fifth load at C...	426 250	127 875	85 000 110 000	85 000 60 000	127 500 140 000	Criterion satisfied. Maximum.
Tenth load at D..	433 750	130 125	110 000	60 000 70 000	140 000 145 000	Criterion not satisfied.

It happens that when the fourth load is at C, the ninth is at D. Hence, Q may include any arbitrary portion of each of these two loads. Thus, if the whole of the fourth load is included in Q, the value of P_1 is 60 000 lbs., while Q may be either 95 000 lbs. or 110 000 lbs. or anything between, depending upon what portion of the ninth load is included in Q. If the whole of the fourth load is included in P_1, the value of P_1 is 85 000 lbs., while Q has any value from 70 000 to 85 000 lbs. The values corresponding to these four limiting cases are shown in Table No. 1. Evidently $P_1 + \frac{n_1}{n} Q$ may have any value between 107 500 lbs. and 127 500 lbs.; and since $\frac{l_1}{l} W = 124\,125$ lbs., the Equation $P_1 + \frac{n_1}{n} Q = \frac{l_1}{l} W$ is satisfied.

Since, in the position of minimum stress the point C is 4.5 ft. from the fourth load and only 0.5 ft. from the fifth, it may be inferred that the stress is greater when the fourth load is at C than when the fifth is there. This may be verified by computing the values of the stress.

The general criterion is satisfied in no other position which seems favorable for greatest stress. The conclusion is that the member has its greatest stress when the fourth load is at C.

(2) *Web Member.*—Consider next the case of a web member, shown in Fig. 3, the dimensions of the truss being as marked in Fig. 8. The influence line shows that the stress is subject to reversal, so that tension and compression must be considered separately. It will suffice to consider the case of compression, corresponding to negative values of the stress-moment.

Inspection of the influence line shows that the load should cover the portion EB of the span; that a load has its greatest effect when at D; and that a maximum compression can occur only when a load is at D (the case of a maximum with a load at A being excluded because the portion of the span AE should be free from loads). It remains to apply the exact criterion.

A trial of successive positions shows that the criterion for maximum stress is satisfied when the second load is at D, but not in any other position which seems favorable for greatest stress. The conclusion is that this position gives the greatest compression in the member. The data for testing three positions are shown in Table No. 2.

From the dimensions of the truss, as shown in Fig. 8, we have $l = 150$ ft., $l_1 = -25$ ft., $l_2 = 175$ ft., $n = 30$ ft., $n_1 = -55$ ft., $n_2 = 85$ ft., $\frac{l_1}{l} = -\frac{1}{6}$ and $\frac{n_1}{n} = -\frac{11}{6}$.

TABLE No. 2.

Position.	W	$\frac{l_1}{l} W$	P_1	Q	$P_1 + \frac{n_1}{n} Q$	
First load at D..	295 000	—49 167	0	0 10 000	0 — 18 333	Criterion not satisfied.
Second load at D.	310 000	—51 667	0	10 000 35 000	— 18 333 — 64 167	Criterion satisfied. Maximum.
Third load at D..	325 000	—54 167	0	35 000 60 000	— 64 167 —110 000	Criterion not satisfied.

(3) *Web Member in a Truss with Parallel Chords.*—In Fig. 4, a negative stress-moment corresponds to tension in the member k. For greatest tension in this member, the influence line shows that only the portion $E\,B$ of the span should be loaded, and that a load must be at D.

Let the span consist of eight panels of 30 ft. each. Then $l = 240$ ft., $l_1 = -\infty$, $l_2 = +\infty$, $n = 30$ ft., $n_1 = -\infty$, $n_2 = +\infty$, $\frac{l_1}{n_1} = 1$, and $\frac{l_2}{n_2} = 1$. The criterion for maximum or minimum stress reduces to Equation (3c), that is,

$$Q = \frac{n}{l}\ W = \frac{1}{8}\ W.$$

It is found that this equation is satisfied when either the third or the fourth load is at D. Both these positions give maximum values of the stress. Between them is a position of minimum stress, which, by applying Equation (3 c), is found to be that in which the third load is 1.5 ft. to the left of D. Comparing this with the two positions of maximum stress, it seems highly probable that the greater of the two maximum values is that which occurs when the fourth load is at D. The numerical data for these three positions are shown in Table No. 3.

TABLE No. 3.

Position.	W	$\frac{n}{l}\ W$	Q		
Third load at D.....	476 250	59 530	35 000 60 000	Criterion satisfied.	Maximum.
Third load 1.5 ft. left of D..	480 000	60 000	60 000	Criterion satisfied.	Minimum.
Fourth load at D.....	488 750	61 094	60 000 85 000	Criterion satisfied.	Maximum.

(4) *Chord Member, Truss with Subordinate Bracing.*—The case represented in Fig. 5 is peculiar in that l_1 and n_1 have opposite signs. The form of the influence diagram shows that for greatest stress the span should be fully loaded, and that a load must be at C for a maximum stress.

Let the span consist of five main panels of 60 ft. each, subdivided into ten panels of 30 ft. each; then $l = 300$ ft., $l_1 = 60$ ft., $l_2 = 240$ ft., $n = 30$ ft., $n_1 = -30$ ft. and $n_2 = 60$ ft. Equation (3a) reduces to $P_1 - Q = \frac{1}{5}\ W$.

Trying successively the positions which appear favorable for greatest stress, it is found that the criterion is satisfied when the thirteenth load is at C, but not with either the twelfth or the fourteenth at C. This is shown in Table No. 4.

TABLE No. 4.

Position.	W	$\frac{1}{5}W$	P_1	Q	$P_1 - Q$	
Twelfth load at C..	765 000	153 000	205 000 230 000	120 000 105 000 95 000 80 000	85 000 100 000 135 000 150 000	Criterion not satisfied.
Thirteenth load at C.....	777 500	155 500	230 000 255 000	110 000 95 000 85 000 70 000	120 000 135 000 170 000 185 000	Criterion satisfied. Maximum.
Fourteenth load at C.....	790 000	158 000	255 000 280 000	85 000 60 000	170 000 220 000	Criterion not satisfied.

When the thirteenth load is at C, there is also a load at D. Since D is a "minimum-point," it is strictly necessary to apply Equation (4) to determine with certainty whether the stress is a maximum or a minimum. We have $W_c = 25\,000$ lbs., $W_d = 15\,000$ lbs., $\frac{n_2}{n} = 2$, and $\frac{n_1}{n} = -1$; and Equation (4) becomes

$$\delta\left(\frac{dM}{dx}\right) = -50\,000 + 15\,000 = -35\,000.$$

Since this value is negative, while M is positive, the stress is a maximum.

The conclusion is that the greatest stress occurs when the thirteenth load is at C.

DISCUSSION.

Mr. Cilley. F. H. Cilley, Esq. (by letter).—The author, in his exposition of criteria for the position of loads causing maximum stress, offers an interesting extension of the well-known rule for loading for maximum moment at a given section of a beam, that the average load on both sides of the section shall be equal. His principle applies to all cases in which the influence line consists of three lines, which include most cases arising in connection with statically determined frameworks. And, in the case of an influence line whose end lines intersect in the region of the middle line, the rule still possesses a simple objective interpretation like that of the earlier moment rule, which applied only to two-line influence lines.

It may, perhaps, be of interest to note some very simple and wholly general facts in connection with influence lines. First, the influence line for the stress in any bar of any framework whatsoever, determinate or indeterminate, consists simply of a series of straight lines, if we neglect secondary stresses. It follows, therefore, that the variation in

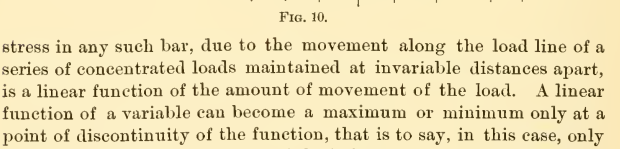

Fig. 10.

stress in any such bar, due to the movement along the load line of a series of concentrated loads maintained at invariable distances apart, is a linear function of the amount of movement of the load. A linear function of a variable can become a maximum or minimum only at a point of discontinuity of the function, that is to say, in this case, only when some load passes an angle of the influence line. For a maximum, it is evident that the angle must be concave toward the zero line in a region of positive influence, convex toward the zero line in a region of negative influence; and for a minimum, the reverse is the case.

A particularly objective and interesting general rule as to the position of loads for maximum influence is found in the following:

Imagine (in Fig. 10), a track of the form of the influence line with positive part down and zero line horizontal. Imagine a series of small wheels, the weights of which are proportioned as those of an actual series of loads, and which are free to move vertically, independently; but horizontally only all together, that is, with constant horizontal spacing. A position of stable equilibrium of this series of wheels on the influence-line track will correspond to a position for maximum influence of the corresponding actual loads with similar spacing. Here,

it is very evident that some load must be at an angle concave from above. The reasons for this rule will readily be perceived. Its immediate source, if the writer's memory is not in error, is an Italian work on graphic statics. Mr. Cilley.

F. C. KUNZ, M. Am. Soc. C. E. (by letter).—The method of influence lines is not so often used as other methods, although it is the simplest way to determine stresses, even for uniform loading, in many cases, and particularly for web-members of all but Pratt trusses, bending moments of pins when simultaneous stresses have to be found, etc. Mr. Kunz.

On page 250 the author gives a general rule for drawing influence lines. For use in special cases, all similar methods derived from the theory of moments offer for web-members but little advantage, as the point of intersection of the top and bottom chords, lies, as a rule, beyond the drawing board. Of all the methods published on this subject, the writer gives in the following the simplest he has yet found. It was first published by Professor R. Land in Constantinople,* and applies to all cases of trusses with verticals and at least one horizontal chord.

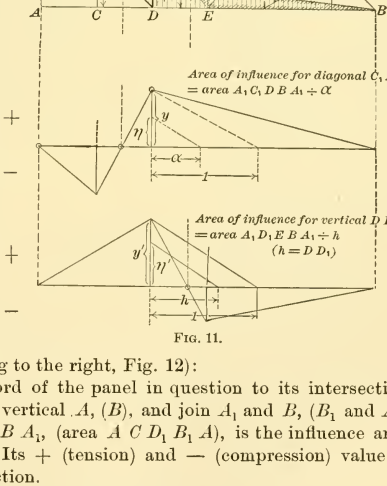

FIG. 11.

For a diagonal $C_1 D$ rising to the left, Fig. 11 (and $C D_1$ rising to the right, Fig. 12):

Produce the top chord of the panel in question to its intersection A_1, (B_1), with the end vertical A, (B), and join A_1 and B, (B_1 and A); then the area $A_1\ C_1\ D\ B\ A_1$, (area $A\ C\ D_1\ B_1\ A$), is the influence area when $\alpha =$ unit load. Its + (tension) and — (compression) value is found by simple inspection.

For a vertical $D\ D_1$, Fig. 11 (and Fig. 12):

Produce the top chord next to the vertical to its intersection A_1, (B_1'), as before; then the area $A_1\ D_1\ E\ B\ A_1$, (area $A\ C\ D_1\ B_1'\ A$) is the influence area when $h =$ unit load.

Professor Land derives these rules in a general way by means of

* *Centralblatt der Bauverwaltung*, 1897.

Mr. Kunz. cinematical principles. The writer gives, in the following, a proof, using the general rule on page 250.

Given four verticals F, A, C and D and a point B (Fig. 13). From B draw any two lines, join $F_1 A_1$, produce this to C_1, join $C_1 D_1$; then the locus of the point of intersection of $C_1 D_1$ and $A_1 B$ is the same vertical z for any two lines drawn from B. Since (Fig. 14), $\frac{m_1}{m_2} = \frac{y_1}{y_2}$, and $y_1 = \frac{a\,m}{l_1}$, and $y_2 = \frac{n_3}{l_2}$; hence $\frac{m_1}{m_2} = \frac{m\,l_2}{n_3 l_1}$, the position of the z-vertical depends only on the relative position of the verticals F, A, C and D and the point B.

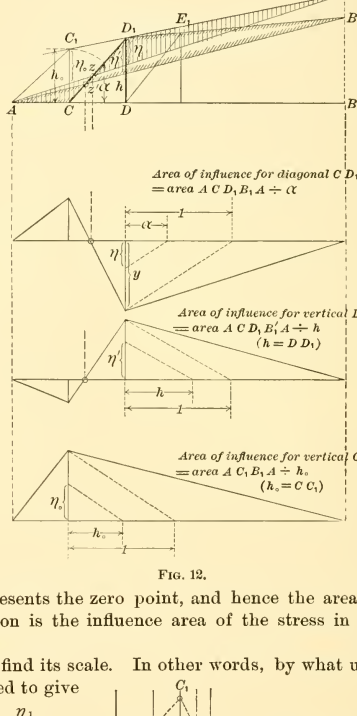

FIG. 12.

In drawing the two lines from B through the skeleton of the truss in such a way that $B\,A$ corresponds to the one and $B\,A_1$ to the other, we see directly that as z_1 lies vertically above z it represents the zero point, and hence the area $A_1\,C_1\,D\,B\,A_1$ in the skeleton is the influence area of the stress in the diagonal $C_1\,D$.

There remains only to find its scale. In other words, by what unit load α must η_1 be divided to give y_1, *i. e.*, $\frac{\eta_1}{\alpha} = y_1$ and $\alpha = \frac{\eta_1}{y_1}$.

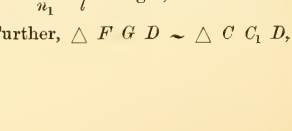

FIG. 13.

$y_1 = \frac{a\,m}{l_1}$, substituting for α its value (see page 250) $\alpha = \frac{l_1\,l_2}{l\,t}$, we have $y_1 = \frac{l_2\,m}{l\,t}$. As $\eta_1 = h_1 - \frac{h_1 l_1}{n_1}\frac{n_4}{l}$ we get, after a short transformation, $\alpha = \frac{\eta_1}{y_1} = \frac{h_1\,t}{n_1}$. Further, $\triangle\ F\ G\ D \sim \triangle\ C\ C_1\ D$,

hence $t = \frac{h\, n_2}{d}$, where d denotes $\overline{C_1\ D}$. Thus we get $\alpha = \frac{h_1^2\, n_2}{d\, n_1}$, and as $\frac{h_1\, n_2}{n_1} = h_2$, we get the unit load $\alpha = \frac{h_1\, h_2}{d}$, which can easily be scaled off in the skeleton. From the proportion $\frac{y_1}{1} = \frac{\eta_1}{\alpha}$ follows the construction of the influence line to any desired scale.

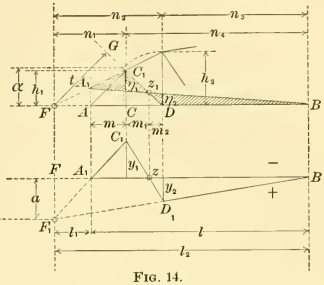

Fig. 14.

For the vertical $D\ D_1$ (Fig. 15) it follows that the point z_1 is the zero point of the influence area $A_1\ D_1\ E\ B\ A_1$. Using the notation in Fig. 15, we have again $y_1 = \frac{a\, m}{l_1}$, where $a = \frac{l_1\, l_2}{l\, t}$, and $t = l_1 + m$; $\eta_1 = h - \frac{h\, l_1}{l}\frac{n_4}{n_1}$. After a short transformation we get $\frac{\eta_1}{y_1} = h =$ unit load, and $\frac{y_1}{1} = \frac{\eta_1}{h}$.

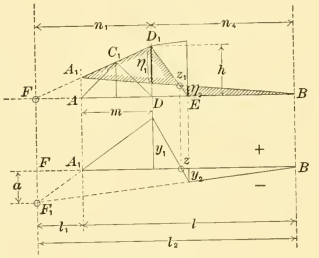

Fig. 15.

Fig. 16 shows the application of the above rule for the diagonals of a half truss of a drawbridge considered as a single span, which will be of particular use in determining the bending moment on the pin D for this condition.

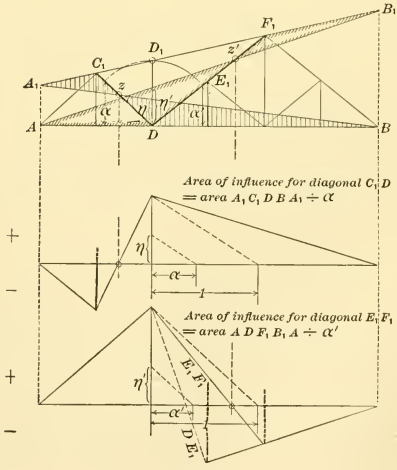

Fig. 16.

Mr. Hoskins. L. M. HOSKINS, Esq. (by letter).—It was no part of the object of this paper to advocate the use of wheel concentrations as a basis of bridge design. This kind of specification has been in general use for a considerable number of years, and, doubtless, will continue to be used for a long time to come, in spite of the efforts of its opponents to supersede it. Even if some different specification should be adopted generally, many engineers of railroad companies probably will think it advisable to apply the wheel-concentration method in estimating the effect of new and heavier train-loads on existing bridges. In view of these facts it seemed worth while to present what appeared to the author to be a material simplification in the treatment of the general problem of computing stresses due to any series of loads whatever.

The members who participated in the discussion at the last Annual Convention of the Society were unanimous in condemning the method of wheel concentrations. They were, however, by no means agreed as to what method should replace it. Thus, the method advocated by Mr. Morison was condemned by Mr. Waddell as complicating matters worse than ever. And, while the majority appeared to favor some form of "equivalent uniform-load," each had his own opinion as to how this should be estimated. That this discussion does not represent the unanimous opinion of those engineers most directly interested in the question is indicated by recent correspondence in *Engineering News*.*

No one supposes that the method of wheel concentrations can represent with exactness the actual distribution of loads upon a bridge. Its advocates merely contend that it comes a step nearer than other methods to representing actual conditions. The truth of this contention is tacitly admitted by most of those who condemn the method. The correctness of every proposed method is tested by a comparison of its results with those of the wheel-load method.

The only real objection to the wheel-concentration method, as compared with other proposed methods, is the difficulty of the computations involved. How serious this objection is can be judged only by one who is thoroughly familiar with the best methods of computation. Wheel concentrations appear to present no serious difficulties to the computer who familiarizes himself with the best methods of dealing with them.

Aside from whatever practical value it has, the problem discussed in the paper possesses considerable mathematical interest. The properties of influence lines given by Mr. Cilley and Mr. Kunz are interesting contributions to the discussion. With reference to the comment of Mr. Kunz, on the general rule for drawing influence diagrams, it may be remarked that when the moment-center falls beyond convenient limits, the ordinates at C and D can be computed readily by the general values given on page 250.

* July 13th and September 7th, 1899.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 860.

PILE-DRIVING FORMULAS: THEIR CONSTRUCTION AND FACTORS OF SAFETY.

By Charles H. Haswell, M. Am. Soc. C. E.

Presented September 6th, 1899.

WITH DISCUSSION.

Supporting Piles.

Experiments on the impact of the ram of a pile-driver, from the ordinary heights of its operation, and deductions therefrom, are not as satisfactory as the importance of the subject requires, as evidenced by the variety in the elements of computation, formulas and factors of safe loads.

Authors upon physical laws, and engineers who have essayed to give a formula for the impact and effect of a ram on a pile, vary not very materially, but in some instances have introduced elements having little or no connection with it, and consequently of no essential value: as the diameter, weight and length of a pile, its mean depression per number of blows, its modulus of elasticity, and the cube root of the height of fall of the ram.

The resistance a pile offers to the blow of a ram is the measure of

its capacity to sustain a stress, wholly independent of its diameter, weight, length, modulus of elasticity, etc.

Thus, the diameter of a pile and the distance of its penetration are not proper elements in the computation, as their effect is embraced in their limitation of the penetration of the pile; added to which the weight of the pile is of consideration only as involving a weight of ram proportionate to its inertia, and the relative elasticity of a pile is of little moment, for when a pile approaches to the penetration of required resistance or refusal, if its head is broomed it is dressed off, and if split or liable to be, it is confined by a ring. In fact, the weight and fall of a ram being proportionate to a pile, all other elements, except that of penetration, are rendered of no consideration, as elements of the computation of effect; and particularly when so great a factor of safety as the varied conditions under which a pile is driven requires, and which is given by various authors of formulas as ranging from 0.083 to 0.33, omitting Weisbach's extreme of $\frac{1}{100}$.

In the following treatment of the subject, a pile is considered as driven to a final depression of 0.5 in. or less, with a level and sound head, as far as the operation renders it practicable; and when the weight of the superstructure to be sustained requires the number of piles to be increased proportionately thereto, this increase objectionably involves the necessity of placing them at less than a proper distance apart, for when they are driven close in a moist or clayey soil, the last one driven will, by the lateral compression, cause the adjoining ones to rise.

In the formulas of Major Saunders, U. S. A., Molesworth, Weisbach, Mason, Trautwine, Nystrom, Wellington and the "Dutch," the final depression or penetration, either in inches, feet or *pro rata*, is given as a divisor. Hence, assuming that the impact of a ram in a given case is 100 000 lbs., and the depression is 0.5 in., $100\,000 \div 0.5 = 200\,000$ lbs., and if it is 0.25 in., $100\,000 \div 0.25 = 400\,000$ lbs.; and in like manner to infinity or positive refusal; and yet the impact would not have increased, as the effect of the blows is not cumulative. The resistance of a pile is not measured by a summation of the several impacts it has received by the fall of the ram, or by the *pro rata* of its depression per number of blows, as by the "Dutch" formula. When a street paver depresses a stone 1 in. with his rammer, and 0.5 in. at a succeeding blow, he has not correspondingly increased the force or

effect of it, neither are the blows of a carpenter in driving a bolt cumulative, for only the last blow gives the measure of the impact on it. Hence, the effect of the blow of a ram, however apparent to observation, is lost to consideration when the one that follows it occurs, as the measure of the last is the measure of the resistance the pile offers, and that resistance is the single sustaining stress or weight which the pile will bear.

The matter, then, is not how the resistance of a pile is obtained, not what the weight of it, not what its inertia or the effect of its cushioning, but what was the power required to drive it to its final depression, or what is the measure of its resistance to stress or to sustaining a weight?

Illustration of the Several Formulas Referred To.

Assume rams having weights of 1 000 and of 2 000 lbs., falling, respectively, 20 and 25 ft., which are taken as exponents of ordinary practice. Mean section of pile, 100 sq. ins.; weight, 1 000 lbs.; surface, 130 sq. ft.; length, 35 ft.; and penetration under last blow, 0.5 in. = 0.043 ft.

(1) *Major Saunders.* $\dfrac{R\,h \div d}{8} = W.$

R representing weight of ram and W safe load, both in pounds; h = height of fall, and d = depression of last blow, both in feet.

(2) *Rankine.* $\sqrt{\left(\dfrac{4\,E\,s\,W\,h}{l}\right)} + \dfrac{4\,E^2\,s^2\,d^2}{l^2} - \dfrac{2\,E\,s\,d}{l} = L.$

E = modulus of elasticity, s = section of pile in square inches, l = length of pile in feet, and L = dead load without further depression. Other symbols as preceding. Factors of safety, 5 to 1.

This formula embraces modulus of elasticity, length and sectional area of piles, all of which, for the causes here given, are held to be an unnecessary complication and of no practical value.

A Second Formula by Rankine. $A\ 1\,000 = L$ at refusal of pile; and when not, $A\ 200 = L$; or a factor of safety of 5.

A = mean area of section of pile in square inches, and L = load in pounds.

(3) *Molesworth.* $\dfrac{W\,H}{8\,D} = L.$

W = weight of ram, and L = safe load, both in hundredweights. H = height of fall, and D = set of pile by last blow, both in inches.

(4) *Mason.* $\frac{W^2 h}{S(W+p)}$ = extreme load.

S = depression by last blow, in feet; p = weight of pile in pounds; other symbols as preceding. Factors of safety, 10 to 4.

(5) *Trautwine.* $\frac{\sqrt[3]{h\ W\ 0.023}}{s+1} \times 2\ 240$ = extreme load. s = depression by last blow in inches; other symbols as preceding. Factors of safety, 10 to 2.

(6) *Nystrom.* $\frac{M^3\ S}{6\ s\ (M+m)^2}$ = safe load.

M = weight of ram, and m = weight of pile, both in pounds. S = fall of ram and s depression, both in inches; other symbols as preceding.

(7) *Wellington.* $\frac{2\ Wh}{s+1}$ = extreme load.

s = depression by last blow, in feet; other symbols as preceding.

(8) *McAlpine.* $80\ (W + 0.228 \surd F - 1) = X$.

W = weight of ram, in tons; F = fall of ram, in feet, and X = load at refusal. Factor of safety, 0.33.

TABLE No. 1.—Recapitulation of Results. Depression in all Cases, 0.5 in.

Formulas.	Factors, as given by the several Authors.	1 000 Lbs. Falling 20 Ft.		2 000 Lbs. Falling 25 Ft.	
		Load.	Safe load.	Load.	Safe load.
		Pounds.	Pounds.	Pounds.	Pounds.
1. *Saunders*.....	8	476 000	59 500	1 190 000	148 750
2. *Rankine*.....	5 to 1	332 125	66 425	646 246	129 249
	5	100 000	20 000	200 000	40 000
3. *Molesworth*.....	8	480 080	60 010	1 199 520	149 940
4. *Mason*.....	10 to 4	273 598	27 360 to 68 399	861 326	86 133 to 21 533
5. *Trautwine*	12 to 2	93 218	7 768 to 46 609	200 592	16 716 to 100 296
6. *Nystrom*.....	6	166 086	27 681	658 434	109 739
7. *Wellington*.....	.....	.....	38 387	.....	95 970
8. *McAlpine*.....	3	83 328	27 776	184 934	61 644
Mean of results.....	5.75	250 554	40 901	645 131	87 271

* A mean by the author.

The varying figures in the fourth and sixth columns, under safe load, represent results as computed by the formulas of different authors, with their factors.

The following formulas were not included in the preceding recapitulation and summation of results, as the factors in the first are held to be too extreme for practical application, and the introduction in the other, of the average penetration per blow, per 100 blows of the ram, is not an element of any value; for, as before stated, the final blow of a ram is the only one which measures its effective value.

Weisbach.—One of Weisbach's formulas embraces elements similar to those of Rankine, with a factor of safety of 10 to 100. A second

$$\frac{w^2 h}{s\,(w + p)} + (w + p) = \text{extreme load.}$$

w = weight of ram, and p = weight of pile, both in lbs.; s = penetration by last blow, in feet = 0.043; other symbols as in the other formulas. Factors of safety, 100 to 10.

"*Dutch.*" $$\frac{B^2 h}{(B + M)e} = W.$$

B = weight of ram, and M = weight of pile, both in lbs. e = average penetration per blow for 100 blows, and h = height of fall, both in inches. Factor of safety, 6.

In the "Dutch" formula, e represents the average depression of the pile per blow, per 100 blows of the ram, which is not only an element measurably impracticable of attainment in practice, but is wholly valueless and difficult of attainment except with the heavy ram and low fall practiced by the authors of the formula.

A recorded case furnishes the following illustration of the formula: B and M, respectively, 2 205 and 2 030 lbs. and e = 0.28 in.

$$\frac{2\,205^2 \times 20}{(2\,205 + 2\,030)\ 0.28} \div 6 = 13\,665 \text{ lbs.}$$

Operating this formula with the elements given in the first of the preceding cases, the result would be 35 715 lbs.

Factor of Safety.—In deciding upon the factor of safety the following elements are to be considered:

The friction of the guides of the leader and of the hoisting line of the ram in the sheave and over the drum (ascertained by experiment with a very heavy ram to be 0.2 ft. of penetration, but with a light ram it would be materially more); the resistance of the atmosphere to the fall of the ram and its cushioning on the head of a pile however squarely it may be dressed off; the want of verticality, both of the fall of the ram, and of the vertical plane of the pile and its consequent

lateral vibration; the inertia and frequent splitting of the pile on a boulder; the vibration, if driven in a trench or with a "follower"; the condition of the soil, whether wet, moist, or dry, that is, if it is wholly dry soil, if the pile is embedded or partially exposed above ground or water, or is wholly immersed in wet soil or surface water, all of which are elements in the determination of the factor.

In this connection is to be considered the subsidence of the soil around a pile which has been temporarily arrested in its driving, the effect of which, in the course of a few hours under favorable conditions of soil, has been observed to approach to that of the resistance of a pile at its last blow.

The following are records of some ascertained sustained weights on piles:

1. At the London Bridge over the Thames,* the piles are reported to sustain 80 tons. Assuming them to have been driven under the extreme condition of a ram of 1 500 lbs. falling 20 ft., their sustaining power in excess of that given by the formula $W\sqrt{2\,g\,h}$ would be 23.94 tons, and $80 \div 23.94 = 3.33$ to 1.

2. A pile driven with a ram of 2 000 lbs. falling 5 ft. supported 47 tons, an excess over the formula of $16 \sim 47 = 31$ tons; and $47 \div 16 = 2.93$ to 1.

3. A pile driven with a ram of 1 700 lbs. falling 16 ft. supported 70 tons, or an excess of $24.3 \sim 70 = 48.7$ tons, or as 2.88 to 1.

4. A test pile by the United States Government, driven by a ram of 910 lbs. falling 5 ft. with a depression of 0.375 in. under the final blow, supported 26.6 tons, or an excess of $7.28 \sim 26.6 = 19.32$ tons, or 3.65 to 1.

The average of results for safe loads, deduced by the several formulas given are, for a ram of 1 000 lbs. falling as given, 40 901 lbs.; a result 2.04 times greater than that given by the formula of an author of highest recognized ability, and 1.46 times less than by another author of similar standing. For a ram of 2 000 lbs. falling as given, the average is 87 271 lbs.; a result 2.182 times greater than that given by the first author referred to, and 1.35 times greater than that given by the other.

In view of this variance, the author essayed to ascertain the difference between the results by the formula, or foot-pounds, and the actual

* "Trautwine's Engineer's Pocket-Book," pp. 643-644.

vis viva of a falling body; and although the experiments were necessarily on a very limited scale, the following results were obtained:

Weight.	Fall.	Velocity.	Impact.	Ratio of Impact to Velocity.
Pounds.	Feet.	Feet.	Pounds.	
1.....	1	8	32	4.0
1.....	1.5	9.5	42	4.3
1.....	2	11.31	52	4.6

Reviewing these and the various other elements submitted, it appears that all the formulas here illustrated, for piles driven under the usual requirements of practice, as to weight of ram, height of fall, character of soil and final depression, are but arbitrary, with factors of safety having the wide range of three-tenths to one-twelfth. They present even the varied conditions of the weight multiplied by the height, by its square, and also by its cube, while that of Wellington presents the exceptional condition of the weight being multiplied by twice the height of the fall; all of them being at variance with the accepted rule, that the velocity of the weight and its resultant impact is as $\sqrt{2\,g\,h}$. Hence, for heights of fall of 16, 25 and 36 ft., the product of the Wellington formula would be as 32, 50 and 72 ft., while the velocity of impact would be as 32, 40 and 48 ft.

Further, that the impact or *vis viva* of a body falling freely, as determined by the experiments here referred to, exceeds four times that given by the formula for its final velocity, which result is verified by the illustrations given of the actual support of piles noted, to which should be added an estimate of the frictional loss of the ram in operation.

Reviewing the elements presented, the following formula is submitted: $\frac{4\,w\sqrt{2\,g\,h}}{F}$, or $\frac{4\,w\,8\,\sqrt{h}}{F}$ = safe load in pounds.

w = weight of ram in pounds, and h = height of fall in feet.

F, or the factor of safety, in consideration of the several losses of effect recited, and especially that of the brooming of the head of a pile, and the driving of piles in trenches or with a "follower," is assumed at from 3 to 6, according to the nature and condition of the soil, the character of the piles and the integrity of their driving.

Applying the formula to elements similar to those given for the

several formulas recited, the results would be, for the illustration of ram of 1 000 lbs. falling 20 ft., with a factor of 3:

$$\frac{4 \times 1\,000 \times 8\sqrt{20}}{3} = L;$$

and for the ram of 2 000 lbs. falling 25 ft., eliminating the numerator 4, and correspondingly reducing the divisors, 3 to 6, to 0.75 and 1.5, the operation is simplified, thus:

$$\frac{1\,000 \times 8\sqrt{20}}{0.75} = 47\,701 \text{ lbs., and } \frac{2\,000 \times 8\sqrt{25}}{0.75} = 106\,666 \text{ lbs.;}$$

and with a factor which is a mean between 0.75 and 1.5 = 1.125, the results would be, respectively, 31 801 and 94 814 lbs., results in accordance with the observation of an exceptionally extensive and varied experience in the operation under consideration.

In further support of the formula submitted, the following is presented:

First.—That at London Bridge, the sustaining power of the piles is reported to be 3.33 times the weight deduced by the ordinary formula for the impact of a falling body; in other cases to have been 2.93 and 2.88 times, and in a test case by the United States Government to have been 3.65 times, giving a mean of 3.1975; to all of which is to be added the greater or less losses of effect of the ram in falling, as herewith detailed.

Second.—That the test pile previously cited, which was driven and observed by officers of the United States Government, the ram weighing 910 lbs., the fall being 5 ft., and the depression of the pile at the final blow being 0.375 in., sustained a stress of 26.6 tons; that the sustaining power of the pile, deduced by the ordinary formula for the velocity of a falling body, would be $910 \times \sqrt{2gh} = 16\,457$ lbs. = 7.34 tons; and that these results of 7.34 and 26.6 are as 1 to 3.62, approximating closely to the 4 assumed in the formula presented.

Third.—That piles driven to sustain the vibratory stress of a bridge under a hammer of 1 200 lbs., falling 20 ft., with a penetration of 0.75 in., enduringly sustained 18 tons each, and by the formula the result is 17 tons.

Regarding the distance from centers at which piles should be driven; in sand or small gravel, this may be 2 ft., but for saturated sand and earth, or in silt, a greater distance is required, as small piles are likely to be disturbed in their position by the driving of a larger one adjoin-

ing them, and in some practice, in determining the capacity of a range of piles, it is proper to reduce the result obtained by the formula, to meet incidental conditions, such as negligence in driving, or in superintendence, the frequent and unobserved splitting or crushing of a pile on a stone or boulder.

The deductions and opinions here given are the result of observations of a very varied and extensive experience in pile-driving, in every variety of soil, encountering quicksand, boulders, submerged cribwork, stone filling, in water, and under the embarrassing conditions met in a trench, when the hammer was necessarily arrested many feet above the designed level of bearing of the piles, and for structures of various capacities and heights, some attaining twelve stories; and whenever practicable, a bench-mark was established and ultimate results observed.

Tenacity of Piles.—The effects of the tenacity were illustrated at Cuxhaven, England, with a fir pile, 1 ft. in diameter at the head, driven 15.7 ft. in fine and drifting dry sand; and which, at the termination of 23 days, required a power of 17 472 lbs. (7.8 tons) to withdraw it; also one of 17 ins. diameter, driven to a like depth, at the termination of 20 years, required a power of 57 120 lbs. (25.5 tons), and at 50 years, 98 000 lbs. (43.75 tons), to remove it.

Friction of Pine Sheet Piles in Clay.—The removal of a coffer-dam after 5 years' existence furnishes the following elements:

Piles of Memel fir, 12.5 ins. square, length 40 ft., driven to an average depth of 18.25 ft., the average superficial area in the soil being 76 sq. ft., the net resistance, after deducting the weight of the piles, etc., was 71 277 lbs., giving a coefficient or factor of 93.8 lbs. per square foot for the bearing of the clay on the inner and outer surface of the piles.

Follower.—Whenever a follower is used, however short it may be, the factor of safety should be increased; and when piles are driven in trenches below the driver, and their final depression below the drop of the ram is so much as to involve the use of a long follower, the extreme factor should be used; and with piles of spruce or like wood, their heads should be ringed as soon as their penetration becomes difficult.

Brooming.—The result of brooming the head of a pile, in consequence of its resistance, as determined by experiment, was to increase

the number of blows of a ram compared with a sound head from 1 to 1.49.

Sheet Piling.—Sheet piling would be much more effective, either in water or wet soil, if constructed of two or more thicknesses of timber laid together and lap broken for half the width of a piece if two are used and one-third the width if three are used, and fastened with locust, or iron blunt bolts, the number of which and their distances apart being determined by the stress of the driving. The off edge of the foot of the pile should be cut or tapered to an extent which will tend to lead and retain it close to the adjoining pile.

Notes Deduced from Operations of Various Authors on Piling.—In wet sand, silt, or any light soil, piles should be driven rapidly, otherwise the subsidence of the material in which they are driven will furnish increased resistance.

In strong or stony soil, a heavy ram with a low fall is better than a light ram and a high fall. If the soil is very strong with stone, the piles should be shod.

Inasmuch as a stratum of light or wet earth may exist below a heavy one, before commencing piling upon a new soil, in order to be assured, not only of its consistency, but also of the depth of such condition, a test pile of small diameter should be driven to as great a depth as practicable.

The support of compressed sand is estimated at 85 lbs. per square foot, and the friction on the surface of a pile at 614 lbs. per square foot.

Finally, in a recent case, in consequence of the foundation of a building of eleven stories being 7 ft. below that of two exceptionally heavy adjacent structures of ten stories each, the walls of which were laid on a concrete bed over wet sand, piling was necessarily arrested at distances of 7 ft. from these adjoining walls, and the spaces bridged with plate-iron cantilever beams. The result, as verified by bench-marks, has proved the sufficiency of the piling under the concentrated stress over the voids, which was spaced and driven in accordance with the formula here submitted.

DISCUSSION.

Foster Crowell, M. Am. Soc. C. E.—The author's experiments upon the impact of falling weights, as recorded in the table on page 273, having been made with a spring-balance, the observed results were the resistances of the spring-balance, exerted through unrecorded distances of retardation of the falling weights, and therefore, if the apparatus had happened to be of a different degree of sensitiveness the values would necessarily have been correspondingly different; consequently the results could not be taken as the absolute values of impact of the falling weights, to arrive at which it would be necessary to determine accurately, not only the resistances developed, but also the distances of retardation. The experiments, therefore, could not be accepted as establishing, even practically, the author's claim, upon which he bases his formula. For, whatever the weight of the impinging body, or the velocity of the impact, there is, in every case, a finite amount of work U, yielded upon the opposing resistances; there is in every such case a certain mean resistance R, overcome through a certain space S, in the direction in which that resistance acts, which resistance and space are such that $R\,S = U$, and $R = \frac{U}{S}$; if, therefore, the space S be exceedingly small as compared with U, there will be an exceedingly great resistance R, overcome through that small space, and *vice versa*. Taking as the values of R, the "impacts" of the table, we find that the corresponding values of S, in inches, are respectively 0.375, 0.410 and 0.461. Thus it is apparent that, had the apparatus been so constructed as to weigh with less movement, the recorded impact might have been very much higher; also, that the movement of the apparatus is analogous to a pile penetration of about 8 ins. under a 20-ft. fall of the ram, in the first case given, but to only about $5\frac{1}{2}$ ins. in the third case, which shows the influence of the friction and lost motion in the apparatus. Mr. Crowell.

Mr. Haswell's formula, as presented, is limited to a penetration of $\frac{1}{2}$ in., and it would be interesting to know what provision the author intended to be made in cases where the observed penetration of the driven pile exceeded that amount, as such cases, in the speaker's experience, constituted the real test of reliability and usefulness of a pile-driving formula. The ideal formula, and the practical one as well, should contain within itself the elements of application to all the cases usually to be met with in practice, and on that account, as well as upon others, the speaker considers the Wellington form of formula preferable to that of Mr. Haswell.

The author has condemned the Wellington formula, including it with those which are arbitrary and at variance with accepted rules,

Mr. Crowell. and specifically because it "presents the exceptional condition of the weight being multiplied by twice the height of the fall," claiming that the true formula should be founded upon the actual *vis viva* of a body falling freely, as represented by its velocity, viz., $\sqrt{2gh}$. This is a misconception, both as to the fact and as to the theory of *vis viva*. The *vis viva* of a body falling freely being Mv^2, and h being the height through which the body must fall to acquire the velocity v, $v^2 = 2gh$; therefore $h = \frac{v^2}{2g}$, and $Wh = \frac{1}{2}Mv^2$; that is to say, the work accumulated in a body falling freely, available for extraneous effect, is expressed by half the *vis viva*, and is always equal to the simple product of its weight multiplied by the distance through which it has fallen. This effect may be expressed in foot-pounds, or inch-pounds, or half-inch-pounds, or by any other combination of weight and fall, but it can never by any possibility exceed itself, and is an immutable feature of the laws of falling bodies applicable to all heights.

Vis viva, according to Mosely,*

"Does not represent a pressure, or force, but the numerical equivalent of the product of a certain number of units of pressure and a certain number of units of path. The one magnitude being of as totally different order from the other as an area is different from a line, and therefore having no common unit of measure."

Impact and *vis viva* are thus totally different things. What enables the descending ram to drive the pile is the energy stored therein, but not all of that energy is available for the purpose, half of it being required to maintain the movement of the ram. That which is available is known as the accumulated work, to be expended upon the sum of the resistances offered by the pile.

In the Wellington formula the principle of accumulated work is the basis; it is expressed in inch-pounds, but as it contains a silent factor of safety of 6, and, as, for convenience, h is given in feet, the accumulated work is written $2Wh$ instead of $W\ 12\ h$. The denominator is the measure of resistance developed by the pile, as indicated by its penetration under the last blow, or settlement, s, also expressed in inch terms. Were it not for the presence of resistances in the pile-driving machine, and other causes tending to oppose the driving of the pile, s would be the full measure of the net resistance, and the quotient would be that resistance; but these various unmeasured opposing forces must be taken into consideration, and they are represented by the constant quantity, 1, in the denominator.

Thus we have the derivation of the Wellington formula,

$$Q = \text{Safe Load} = \frac{2\ Wh}{s+1}.$$

* "Mosely's Mechanics of Engineering," p. 589.

Mr. Crowell.

Comparing this with Mr. Haswell's formula, which is

$$\text{Safe Load} = \frac{8\ W\sqrt{h}}{1.125},$$

we perceive that for the $\frac{1}{2}$-in. penetration, to which the latter applies, it becomes $Q = \frac{2\ W\ h}{1.5}$, and if we assume a fall of 16 ft. we shall have

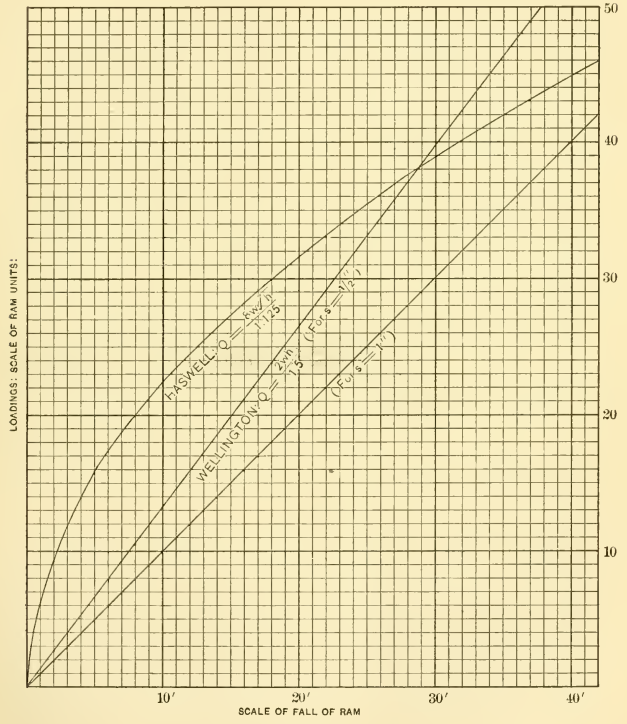

FIG. 1.

by the Wellington, $Q = \frac{32\ W}{1.5}$; and by Haswell, $Q = \frac{32\ W}{1.125}$; results differing only in the factors of safety. That is to say, with the same actual factor of safety the results for the two formulas would be identical for the assumed fall of 16 ft.

Mr. Crowell. The assumed factor of safety in the Wellington formula is 6, while the assumed factor in the Haswell is only $4\frac{1}{2}$; actually, the above Wellington expression gives $Q = 21.333\ W$, and the Haswell $Q = 28.444\ W$.

The two will be found to give absolutely the same working result with a fall of 28.45 ft. A study of the diagram, Fig. 1, of the characteristic plottings of the respective formulas may be instructive. The Haswell formula is the equation of a parabola and Wellington's is a straight line; the abscissas are the heights of fall of the ram and the vertical ordinates the safe loads expressed in units of ram weight. It will be noted that for low falls the Haswell formula gives relatively great safe loads and for high falls relatively smaller loads.

Considering that the highest value given by the Wellington line has a theoretical factor of safety of 6, it is apparent that there is no advantage in the drop of the Haswell curve at the higher falls. On the other hand the exceeding great values, relatively, given by it for the low falls show its inconsistency and the danger of its use by the inexperienced. It will be seen that for a fall of only 2 ft., the indicated safe load is one-third as great as that indicated for a fall of 18 ft., nine times as great.

The lower of the two inclined straight lines in the diagram is introduced to show the Wellington values when the penetration is 1 in. It will be observed that the margin of safety is somewhat decreased, so that, strictly, the supplementary quantity in the divisor should not be a constant, but a variable, as was pointed out by the speaker in the discussion on his own paper on this subject, presented before the Society several years ago.* There can be no question, however, that for ordinary penetrations occurring in practice the Wellington formula is amply safe, and, in the speaker's opinion, its simplicity and ready applicability entitle it to preference.

Mr. Gould. E. Sherman Gould, M. Am. Soc. C. E.—It is needless to say that, in the speaker's opinion, this paper has great practical value. The long experience of the author in the kind of work described, and his mature judgment, give importance to his utterances. He exposes the vast array of irrelevant elements introduced by many authorities into a problem which, from its nature, does not admit of mathematical refinement, and presents a formula which, in the speaker's opinion, has much to recommend it. Its chief peculiarity is in assuming the constant value of $\frac{1}{2}$ in. for the final set, instead of a varying one. That is to say, his formula assumes what amounts practically to "refusal," or the absence of further penetration after the last blow of the hammer.

The problem which confronts us in pile-driving formulas, is to find

* "Uniform Practice in Pile-Driving," *Transactions*, Am. Soc. C. E., Vol. xxvii, pp. 108, 596.

Mr. Gould.

the true relation between the effect of the sudden impact of a falling heavy body, and the steady pressure of a quiescent one. No such relation has yet been discovered, and all that can be done is to seek an empirical expression which will be a safe, practical guide. The formula quoted by the author as "Wellington's," is probably the one which has most endeared itself to American engineers from its simplicity and generally safe results. Its denominator is a binomial expression containing the variable set of the pile, or its final penetration, plus unity. In this form it is relieved of the absurd results which would otherwise ensue from taking very small values for the set, which absurdity would reach its climax if we should assume absolute, or zero set, when the formula would give an infinite bearing capacity to the pile, no matter how light the hammer or how low the fall. This impossible result is avoided by using $s + 1$ in the denominator.

Wellington's formula, however, like all others recognizing movement or set in the pile under the last blow, fails to meet the requirements of the problem. It is merely the statement of the equality between the work stored in a heavy body falling through a certain height, and a force, called the "safe load" moving through a relatively short distance called the "set." There is no quiescent pressure involved, nor can there be. Work in the hammer can only be offset by work in the pile, and work implies movement and not rest. The "safe load" is not, therefore, according to these formulas, the steady pressure which the pile could bear at the moment of receiving the last blow, but the equivalent pressure which would at that moment force the pile down the distance of the final set.

After receiving the blow it may be assumed, on empirical principles, that the pile could bear that weight without any further movement.

These considerations appear to the speaker to meet the question: What formula can be used safely in the case of a set of considerable amount, 6 ins., for instance? The answer appears to him to be: None at all. He considers that, in order to use any formula successfully, the piles, in all cases, should be driven to a final set of not more than 1 in. If a pile 40 or 50 ft. long continues to go down as much as 6 ins. under the hammer, then, to apply a formula. the weight or fall should be reduced until only 1 in. set is produced, and its bearing capacity then calculated by whatever formula may be preferred. If the final set is taken at 1 in., then Wellington's formula reduces to Wh.

In regard to the Wellington formula, which the speaker has always been inclined to favor in the above modified form, it must be understood distinctly that, like all empirical expressions, its use applies only to a very restricted range of variation, such as is ordinarily met in practical pile-driving. That is to say, it is not to be taken as a

Mr. Gould. general proposition in applied mechanics. If we so regard it, we shall be involved in the practical absurdity of admitting the effect upon the pile to be the same whether a 50-lb. hammer and 1 000 ft. fall were used, or a 5 000 000-lb. hammer and 0.01 ft. fall.

If specifications called for a hammer weighing 2 000 lbs. and a fall of 20 ft., giving 40 000 foot-pounds, and it were found that only a 1 500-lb. hammer were available, it might, under the circumstances, be admissible to use the lighter weight with a 27-ft. drop, but not one of 1 000 lbs. and a 40-ft. drop, the effect of which would be quite different from that intended.

Mr. Haswell uses the square root of the height, instead of the height itself, in his formula. The speaker is inclined to believe that this is correct, as regards the work practically done by the hammer, and with the distinct understanding that the formula applies, like the Wellington, only to comparatively small differences of fall. The formula also contains a factor of safety, varying from 3 to 6. Taking the higher value, and using round numbers, it will be seen that Mr. Haswell's formula reduces to

$$L = 5\ W\sqrt{h},$$

in which L = the safe ultimate load, expressed in the same unit as the weight of the hammer.

It has already been shown that Wellington's formula, for inch refusal, reduces to

$$L = W\,h.$$

Within the ordinary range of height of fall and admitting a refusal of 1 in. in both, the two will not differ very widely, and the speaker is inclined to believe that Mr. Haswell is correct in minimizing the effect of height, particularly as this tends to encourage the use of heavy hammers and low drops.

The whole subject of pile-driving is one of judgment rather than of mathematics. Mr. Le Conte has mentioned some of the curious changes in the set of a pile brought about by time, according to the character of the ground penetrated. It seems, however, hardly worth while to consider the case of such materials as sand, gravel or hard-pan. Generally, when such hard formations are reached, pile-driving is stopped and a case of bearing piles is presented. Soft material under or above water is what we are most concerned with, and it is a matter of common observation that piles driven in such material become so bound by skin friction, that, after 24 hours of repose, their refusal becomes much greater than during the original driving. This seems to be true to a marked degree in the case of jetted piles. In such material, piles should be sent down as rapidly as possible by a continuous volley of quick blows. This would work havoc with our formulas, unless the pile driver were sent back again next day to test the piles for final set; but the practical result would be superior.

Mr. Howe. HORACE J. HOWE, M. Am. Soc. C. E.—The results of the experiments in Brooklyn by the late W. J. McAlpine, Past-President, Am. Soc. C. E., have been published in many papers. One of his conclusions is, that the effect of increasing the fall of the ram is to increase the sustaining power of the pile in the ratio of the square root of the fall.

Another conclusion is, that by adding to the weight of the ram the sustaining power of the pile was increased by $\frac{7}{10}$ to $\frac{9}{10}$ of the amount used to the ratio of the augmented weight of the ram.

As a result of Mr. Haswell's experiments with 1-lb. weights, he presents a formula for the safe bearing power of a pile, in terms of the weight and fall of the hammer, an assumed penetration of $\frac{1}{2}$ in. or less (which does not appear separately) and a factor of safety. As in other instances, this safe load is found varying directly with W, and a function of h and inversely as F; and there seems to be no new term added to make the formula distinctive.

Passing from the question as to whether the experiments have been interpreted properly, to the general question of the application of a formula to common practice, there seems to be the necessity of defining, as far as possible, the limit of the reliability of any formula.

All that can be claimed for any formula for the safe load on a pile is governed by the condition of the ground at the time of driving.

If we follow the career of a pile after the formula has left it, we enter into the domain of history, and it is a question of how far into history we care to go. If we wish short-time tests, say a week, or a year, we may arrive at certain conclusions. If we wish long-time tests, say 40 or 50 years, we may arrive at certain other conclusions, or our successors may. But a formula is only a neat expression for a very short-time test indeed, and the emphasis, at the present time, should be placed on the deleterious action of long-time, and more or less continuous, forces.

For instance, take the movement of a railroad train near a building. The speaker has sat in a frame house 1 000 ft. from the track, and has been astonished many a time at the noise made by the structure, due to the vibration transferred through dry loamy soil to the masonry foundations, and thence up to the top story.

Again, another example is a factory on Fort Point Channel, Boston, in which the movements of the piles are seen plainly when the machinery is in motion. A long pendulum is set up inside the building to chronicle the changes. The building is new, and the blue clay is pretty reliable, and, therefore, the settlement, so far, has been slight.

Sometimes we see or hear of a layer of concrete 5 or 6 ft. thick going into a foundation, without, perhaps, being able to find the man who figured the exact thickness required. In the light of these principles, and with a pile foundation, it may not be altogether a waste of money.

Mr. Howe. Then, water is another long-time agent, tending not only to convey lubricants to the pile, but to move the adjacent soil perhaps laterally.

The matter of incident decay, too, is of the widest interest. It involves a knowledge of the proper elevation above low water at which to cut off the pile or to limit the grillage in different soils and for different purposes. The determination of this limit is still in the experimental stage, and depending on the years as an umpire of opinions.

Then another point, beyond the reach of the formula, is the relative supporting power of one pile, and of a mass of piles; or the relative supporting power of a small and of a large surface. This is also in the experimental stage.

Meanwhile, the pile, with its never-sleeping load, takes advantage of any errors of judgment on the part of the engineer, and slips down and down and never rights itself. So that, for work which is to be permanent, the emphasis should be on the long-time experiments.

Experiments to the ultimate, of any kind, are rare, and it may be the proper time to say that it is scarcely creditable to the profession. Recently, in a room full of engineers, the question was asked as to ultimate experiments, and there was no answer.

The profession is indebted to Mr. Haswell for again bringing this condition of things to its attention.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is much interested in this subject, and is pleased with the manner in which it is presented. The author is correct in his criticism of existing formulas. The true cause for this extraordinary discrepancy in computed results, as determined by standard formulas, is to be traced to the fundamental fallacy of assuming that the resistance of the soil to the penetration of the pile is constant, for any given depth, during the time the pile is being driven, and that this condition remains constant after the pile is driven. Both of these assumptions are absolutely untenable. Hence the writer has considered the problem as being largely beyond the reach of mathematical formulas which demand rigid facts as a foundation, and this rigidity does not exist. The following notes, based on personal experience, will go to show the absurdity of relying upon the computed results as obtained from the use of the ordinary standard formulas.

Case 1. Sandy Soils.—Soon after driving begins, a cone of resistance forms at and near the foot of the pile, and the material comprising this cone becomes more and more compressed as the pile descends, the penetration at each blow becoming less and less until it reaches a final minimum, of, say, 0.5 in., when the driving is completed. Now, wait 24 hours or more, and, by way of experiment, begin driving a second time. It will be observed that the same pile now penetrates 4 to 5 ins. at the first blow. What does this mean? It simply means that the compressed material surrounding the pile has had time to dissi-

pate, or readjust itself, so that the soil has resumed its normal condition, such as existed before any driving was done. It would appear, therefore, that, in sandy soil, the penetration at the last blow is a very dangerous and misleading element to figure on, for the reason that it is dependent upon a temporary, abnormally compressed condition of the soil, which is always short-lived, and cannot be depended upon as a reliable function in any formula. It is evident that, in sandy soil, the Dutch formula, by *pro rata* of number of blows, is, for obvious reasons, preferable to the final-penetration formulas. Mr. Le Conte.

Case 2. Plastic Muds and Clays.—The physical conditions are entirely different from those in Case 1. It is true that the penetration at each successive blow becomes less and less until the driving is completed, that at the last blow being, say, 3 ins. Now, as before, wait 24 hours or more and begin driving the same pile a second time. It will be found that the hammer, after repeated blows, cannot start the pile at all. In this case, then, the penetration at the last blow, 3 ins., is no index whatever as to what load the pile will withstand. The blow necessary to start the pile after standing at rest for 24 to 48 hours is the better test. This curious physical property of the plastic muds and clays has been properly likened to the similar results obtained by forcing a pin into a solid india-rubber ball. The same force which pushed it in will pull it out again if it is pulled out immediately; but, after waiting 24 hours, the force required will be five times as great.

Case 3. Hardpan and Sandy Clays.—In this material the most perplexing results of all are to be found. The writer has never seen any two hardpans which developed the same results. The ultimate result depends, not only upon the percentage of clay and sand in the hardpan, but also upon the solubility of the clay when brought into contact with the local ground-water. Where piles are driven through ground-water overlying hardpan, the water invariably follows each pile down as it penetrates the ground, and softens the clay in contact with the surface of the pile, and often practically destroys all lateral friction. In such cases the only point of support is at the end of the pile, and the overhead load will be supported by a cluster of columns (piles), the heights of which are equal to the length of the piles.

Finally, there is no doubt about the fact that the best way to test the bearing power of piles, in any given ground, is to drive experimental piles to the required depth, then load them with pig-iron and observe the effects after long standing, and thus find by trial their ultimate resistance. This is not always practicable, in which case the next best way is to drive experimental piles to the depth required or specified, then let them rest for 48 hours or more and drive them a second time, and note the drop of the hammer necessary to start the pile and the first penetration in inches. The results thus obtained represent more nearly the resistance of the soil in its normal condition.

Mr. Haswell. CHARLES H. HASWELL, M. Am. Soc. C. E. (by letter).—In response to the remarks of Mr. Crowell, on the results and deductions of the author's experiments to ascertain the relative impact of falling weights from different heights, it was not essayed, nor was it held to be at all practicable, to determine exact measures of impact with an instrument possessing such retarding elements as a spring-balance, supplemented with a detent. It was essayed only to ascertain the relation and ratio of the impact of the weights from different heights of fall, and as like causes produce like effects, the relation of the results, satisfactorily determined by repeated operations, were just as conclusive as if obtained with the exactness of the requirements of normal results.

Regarding the final penetration of 0.5 in. being assumed as the base of the formula submitted, it was taken, inasmuch as it is not only the ordinary limit of practice, but it is a sufficient refusal and the practically proper one; as the difference between it and a less depression is not only difficult to observe, but the attainment of it is of little relative value with the usual factor of safety, and when effected, it results frequently in the splitting of the piles; further, if it is different, the factor can be altered to meet the conditions, and in any formula, if the final penetration is not a fixed element, computation of the capacity of a pile is emasculated, and the judgment of the driver from visual observation is the only result whereby to assign its capacity.

A mean factor of 4.5 is assumed, and when it is considered that the resistance of a pile in one hour after being driven, is, by the subsidence of the soil around it, very much increased, it is held to be a full measure of safety. In moist sand, with a rough pile of oak or pine (not spruce), a factor for a quiescent load is, in the author's opinion, practically unnecessary, and this opinion is based upon exceptional opportunities to observe the effect of such conditions.

It is not the author's purpose to discuss the application of *vis viva* in this case, it is preferred to give observed results. The resistance an instrument opposes to a falling weight is assumed as a measure of its force, whether it is a spring-balance or a supporting pile.

In further support of his remarks, Mr. Crowell submits a diagram illustrative "of the characteristic plotting of the respective formulas" of Wellington and the one submitted, and that "it will be seen that for a fall of only 2 ft., the indicated safe load is one-third as great as that indicated for a fall of 18 ft., nine times as great." Now, as the formula gives but three times for the 2 ft., and as $\sqrt{2\,g\,18}$, is just three times that for 18 ft., the formula agrees with the illustration, and the result is not nine times, as asserted.

Mr. Gould is inclined to believe that the use of the $\sqrt{H}$ is correct. His opinion, from his capacity and great experience, is entitled to much consideration.

Mr. Howe recites that by the published experiments of Mr. McAlpine, the sustaining power of a pile is increased in the ratio of the square root of the fall of the ram. This also is in support of the formula used, and is wholly at variance with the formula of Mr. Wellington. Mr. Haswell.

Mr. Le Conte's deduction and experience, regarding the second driving of a pile after an interval of a day in sandy soil, is at variance with the author's experience, and as to the value of the Dutch formula, the author fails to recognize the utility of considering the number of blows of a ram, when the effect of the final blow or impact of it is the measure of the resistance of the pile. The tenacity of a nail in wood is greater when driven at one blow than by a number of them, for with one blow there is but one resilience of the disturbed fibers of the wood, which resistance is the element of the tenacity of the nail, and a repetition of blows is a repetition of the resilience and consequently a weakening of it.

"Molesworth on Pile-Driving" gives a table of the force of a blow of a falling weight $= V\ W$.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 861.

THE ARTIFICIAL PRESERVATION OF RAILROAD TIES BY THE USE OF ZINC CHLORIDE.

By WALTER W. CURTIS, M. Am. Soc. C. E.

PRESENTED MAY 17TH, 1899.

WITH DISCUSSION.

Over a year ago the author became interested in the question of the present commercial value of the zinc-chloride process for preserving timber, and particularly ties; with the result that he began an investigation to determine for himself the value of the treatment and the occasion for it in the United States.

Since the report made to the Society by its Committee* in 1885, there has been but little published on the subject, and that has been in pamphlet form, making it extremely difficult, if indeed possible, to secure recent and accurate data on the subject. Probably many will be surprised to learn that during the last 12 years something like 10 000 000 cross-ties have been thus treated, and that during the present year there will probably be 1 500 000 ties treated.

It is hoped that this paper may to some extent add to the knowledge on the subject and place in accessible form that which has already existed for those who knew where to find it.

* *Transactions*, Am. Soc. C. E., Vol. xiv, p. 247.

Ties in General.

Any investigation of the timber resources of the country is bound to be exceedingly disappointing, on account of the impossibility of securing any complete, or even correct partial data.

The Government, through the Department of Forestry, has investigated the subject, and has collected much information as to the supply, the consumption, the characteristics of the material and the best methods of using and economizing American timbers. With reference to the special subject of railroad ties, the Department, in 1882 and 1883, by means of circulars, collected data from 283 different corporations, representing over 70 000 miles of track. This data covered the size and number of ties used per mile, the kinds of timber used, its durability and cost, with general information as to the sources of supply, etc. In 1886 inquiries covering somewhat similar ground, resulted in replies from 35 representative corporations, operating 46 000 miles of track. The first indicated the average life of ties to be as follows: White and burr oak 7 years, cypress 9 years, redwood and cedar 11 years, and for other woods generally 5 years. In the later report it is stated:

"While no general rule can be laid down as to the lasting quality of timber imbedded under the track and used in bridges and trestle work, subject, as it is, to the endless varieties of conditions of soil and climate in different sections of the country, nevertheless, it may be safely assumed that 7 years is the limit of the average usefulness of ties of the best kinds of hard wood, and 4 years for the soft woods."

The 46 000 miles of road, covered by the 1886 report, used in renewals, the preceding year, 16 700 000 ties, at an average cost of 35.6 cents each. "The average number of new ties required annually for each mile, to replace decayed ones, is 365; the average duration of ties, 7.2 years."

In the report on metal ties and the preservative processes for wooden ties, prepared by E. E. R. Tratman, Assoc. M. Am. Soc. C. E., under the direction of Mr. B. E. Fernow, Chief of the Division of Forestry, and issued in 1894 as Bulletin No. 9, the situation is summarized thus:

"Briefly recapitulating statistics as given in the previous bulletins, it may be stated, that with an average of 2 500 ties per mile, the 230 000 miles of track, including second, third, fourth and side tracks, represent 575 000 000 ties in service. Renewals require an-

nually upwards of 75 000 000 ties" (corresponding to $7\frac{2}{3}$ years of life) "new construction 13 000 000 ties, or the total annual consumption in round numbers of 90 000 000 ties, equal to 450 000 000 cu. ft. of forest-grown material, to which some 60 000 000 cu. ft. for bridges and trestle work must be added, making the total consumption for railway purposes exceed 500 000 000 cu. ft. This requires the annual culling of the best timber from probably more than 1 000 000 acres, and the annual product of at least 50 000 000 acres in good condition, or more than 10% of the present forest area of the United States reserved for this one use alone."

Benjamin Reece, M. Am. Soc. C. E., demonstrates* conclusively the very largely increased relative importance of tie renewals as compared with rail renewals; the expenditure for ties on various roads having increased relatively to rails in the years 1880 and 1890 nearly 300 per cent. Most of this remarkable increase was undoubtedly due to the decreased cost of rails per ton; and the data are lacking in Mr. Reece's paper to determine how much, if any, is properly attributable to increased cost per tie or to decreased life. The increased importance to the railroads of the expense of tie renewals, however, is not affected, whatever the reason; and the evidence of the urgent need of study and effort to reduce the cost of such renewals is sufficient.

It is difficult to learn much about the history of the ties on many American roads. A rail carries with it the name of its maker, the year made and a guaranteed life. The tie is simply a stick, looking much like any other, with no marks on it and no record anywhere. The officials know that the first ties were originally laid during a certain year; that perhaps these were largely replaced when new rails were put down at a later date; that some have been renewed each year since; and generally that is all that can be accurately stated. Some roads can do more, and can tell how many ties have been replaced on each section each year; but if an attempt is made to ascertain the average life of a certain kind of tie, or of all kinds in the main track, it will almost invariably result in disappointment. In certain instances there may be particular stretches of road where for special reasons the roadmaster remembers more definitely how many ties have been removed. This is generally where the ties have been remarkably durable, or else short-lived, and, based on that recollection, the life of that particular kind of tie is apt to be given as longer or shorter than a general average would justify.

* "The Increasing Cost of Railway Tie Renewals." *Transactions*, Am. Soc. C. E. Vol. xxvii, p. 640.

The trouble is that there has never been, nor is there now, any generally accepted method of marking ties; and indeed there is no particular advantage in knowing just how long any single untreated tie has been in place. Presumably the best obtainable of the kind are bought, and when they are destroyed more must be purchased. The kind selected has usually been the best obtainable from the adjacent land.

It is hoped that the discussion of this paper will bring out the records of careful determinations of the life of various woods used as ties.

It may be concluded that at present the only information regarding the probable life of ties, free from the very large element of personal opinion, to be secured from American roads generally, is to be obtained by taking the total tie renewals per mile of road per year, averaged over, say, 5-year periods. This will indicate the average number of annual renewals per mile, and whether the average life of the ties is varying. Covering sidings as well as main tracks, however, such information is misleading, as regards the life of a tie in the main line, must be carefully interpreted in the light of the particular conditions for each road, and can be considered as indicative only. To compare one road with another, the conditions must be alike; about the same relative proportion of siding and main track; of hard and soft wood, and of tie-plates used; and the same climatic conditions must obtain. As indicative of the latter, J. F. Wallace, M. Am. Soc. C. E., Assistant Second Vice-President of the Illinois Central Railroad, states that white oak ties on his lines south of the Ohio River have an average life of from 4 to 6 years, depending upon the height of the land where the timber was grown, the season when cut and the kind of ballast.

North of the Ohio, in central and southern Illinois, the average is from 7 to 9 years, and in northern Illinois and Iowa it is from 9 to 12 years.

Through the kindness of the officers of some roads and from an examination of the annual reports of others, the author has secured certain data on renewals, which are of sufficient interest to warrant preservation. Mr. Wallace states:

"The average number of ties used each year on the Illinois Central Railroad for the past 18 years has been, approximately, 300 to the mile. The annual number used per mile, however, has varied from as low as 221 to as high as 434. I estimate it will require an average

of 350 ties per mile per annum, taking our line as a whole, to keep up normal repairs, on the basis of 3 000 ties per mile of track."

Generally speaking, white oak ties are used north of Memphis, and cypress ties south thereof.

E. A. Handy,* M. Am. Soc. C. E., has given the annual renewals per mile for his road for the past 15 years, showing a variation from 174 to 359 per mile. The average per year for the 5 years from 1883 to 1887 is 310; for the 5 years from 1888 to 1892, 317; from 1893 to 1897, 209. The only explanation obtainable for this very remarkable decrease is the reported construction of a large amount of second track in 1892, 1893 and 1894, resulting in increased mileage without increased tie renewals. However, as the total trackage of all kinds in 1891 was 2 512 miles, and in 1897 was 2 651 miles (an increase between these years of less than 6%), the one-third decrease in renewals is hardly accounted for. The extensive preparations made prior to 1893 for the demands of traffic during the Exposition at Chicago may have had something to do with it. As evidence that it was not due solely to extreme economy during the poor years just past, it may be stated that the renewals for 1897, while in excess of the four previous years, were only 245 per mile. White oak ties are used principally.

The annual reports of the Chicago and Northwestern Railroad give the average annual renewals for the years 1883 to 1887 as 206 (not exact, as the length of side-tracks for 4 years was estimated), 1888 to 1892, 216; 1893 to 1897, 207. During these years the track mileage increased from 4 100 to 7 057, the increases being made by yearly additions of reasonable uniformity. So far as an indication of the real average life of ties is concerned, the figures are deceptive.

An average renewal of 210 ties corresponds to a life of $13\frac{1}{3}$ years, assuming 2 800 ties to the mile, and, on the mileage of 1897, to a total number of ties renewed of 1 481 970. If the 13 years in fact represented the average life, and no allowance was necessary for the ties destroyed earlier on the extensions, before the average life had been reached, this number of renewals should correspond approximately to that of the mileage in 1884, which was about 4 400. As this gives a renewal per mile of 337 ties instead of the actual 210, it is evident that this latter figure is deceptive and is due to the rapid increase in the length of the road.

* Chief Engineer of the Lake Shore and Michigan Southern Railroad.

Further trial computations indicate to the author that the probable average life is about 10 years, corresponding to an average renewal rate of about 280 ties per year.

Mr. J. E. Blunt, Chief Engineer, states that the ties in all their principal main lines are white oak, now of a decidedly poorer quality than formerly used, from which they get 8 to 9 years' life. The balance of their ties are cedar, giving 12 to 14 years' service. The ties, per mile, average about 2 700.

R. Montfort, M. Am. Soc. C. E., has kindly furnished statistics for the Louisville and Nashville Railroad, from which the following results have been deduced:

Average renewals per mile for 15 years prior to 1898	360
" " " " 10 " " " "	342
" " " " 7 " " " "	334
" " " for the year 1896–97	398
" " " " " 1897–98 as recommended	375

Noting that the figures for the renewals for the 10 and 15-year periods include only those divisions which have been built that length of time, on which the amount of new track laid is presumably quite small, it is fair to assume the average life of ties on this road to be about 8 years.

On the Chicago, Milwaukee and St. Paul Railroad the record, prior to 1889, was not readily obtainable. Beginning with that year's annual report the renewals each year to 1897 were, 280, 210, 204, 228, 245, 245, 266, 262 and 242. The average of these is 243, corresponding to a life of about 11 years, the average ties in use per mile of all tracks being about 2 700. During these years the mileage increased from 6 810 to 7 747, an amount probably not sufficient to affect greatly the apparent life of the ties. At present about 50% of new ties purchased are white oak, and about 45% white cedar. In past years a considerably larger percentage has been white oak.

Through Thomas Rodd, M. Am. Soc. C. E., the statistics for the Pennsylvania Railroad, West of Pittsburg, have been furnished by Mr. Charles Watts, General Superintendent. The average life of various woods used for ties is:

Oak, 9.2 years; hackberry, 4 years; tamarack, 5 years; cedar, wild cherry, honey locust and black walnut, 8 years.

The method of determining these periods is not stated. The tie renewals since 1883 have varied from 168 to 363. The average by 5-year intervals is: 1883 to 1887, 236; 1888 to 1892, 289; 1893 to 1897, 212. During these years the cost of ties has increased about 10%, and the quality has deteriorated. The length of haul in the woods has increased an average of 6 miles for the system, indicating the exhaustion of the nearer lands.

Mr. J. B. Berry has furnished data for the Fremont, Elkhorn and Missouri Valley Railroad, of which road he was Chief Engineer until quite recently.

The figures do not extend back beyond 1893, but they are remarkable, both for their extreme smallness and the excellent system indicated. The entire renewals for 1 344 miles of main track, none built later than 1888, and some in 1868, were 470 per mile for the years 1893 to 1898, inclusive, or less than 80 per mile per year. The renewals figured for 1898 were those recommended to be used. This very small number is due to various causes : The location of the line in an extremely dry section, the rigid economy required during the period of commercial depression, the light traffic and the use of large numbers of cedar ties under their most favorable conditions. A considerable part, however, is undoubtedly due to Mr. Berry's systematic and constant inspection of the ties, as requisitioned for and, when removed from the track, as destroyed. Each year tables similar to those shown in Forms A and B have been prepared, giving the data for every portion of the road.

FORM A.

Location.	Miles of main track.	Year road built.	Average number of ties per mile of main track.	Assumed average life of ties in years.	Making average per mile per year for main track.	Number ties requested per mile for 1898 main track.

FORM B.

Location.	Miles of main track.	Year road built.	Renewals per Mile.					Proposed for 1898.
			1893.	1894.	1895.	1896.	1897.	

All ties removed during the year are piled up and not destroyed until inspected by the Chief Engineer, and to this *post mortem* Mr. Berry attributes most of the credit. Presumably he claims no originality for either the method of tabulating the facts or the final inspection. Mr. Reece, in the paper previously referred to,* states that he inaugurated the same system on the Lake Shore and Michigan Southern Railroad with great benefit; but, as the author has found sufficient evidence of the need of such systematic care, he feels justified in presenting Mr. Berry's method to the Society.

None of the roads mentioned has used treated ties more than experimentally, if at all. The figures given for renewals, excepting in the last instance, are per mile of all tracks, both main and sidings, and consequently correspond to a longer life than is obtained in the main track. This accounts for the discrepancy between the life of ties given by the average renewals, and that previously stated by the various roads, as the actual life of particular woods in the main track service. The average annual renewals for the past 15 years, indicating very clearly the climatic effect, are:

Pennsylvania Railroad, West of Pittsburg.....245;
Lake Shore and Michigan Southern.....280;
Chicago and North Western .....280 (corrected for extensions);
Chicago, Milwaukee and St. Paul.....243 (for 9 years only);
Illinois Central. .....300 (350 on basis of 3 000 ties per mile);
Louisville and Nashville..... ...360.

Ties perish by the mechanical destruction of the fibers under the rail; splitting by seasoning and cutting out of fiber by respiking; and by natural decay. In its normal condition, to resist these three influences, a timber is required which shall have sufficient hardness to resist the mechanical injury; a high degree of coherence between fibers ; and a sap of such chemical composition as will offer the least encouragement to the development of the fungoid life which destroys the fiber.

The destruction of the fibers under the rail is generally attributed to the cutting and abrading action due to the deflection of the rail and

* *Transactions*, Am. Soc. C. E., Vol. xxvii, p. 640.

tie and the longitudinal movement of the rail as the load passes over it; and the direct compression of the timber is considered as too small to affect it. If a 60-lb. rail has a bearing on the tie of $4\frac{1}{2}$ x 8 ins., or 34 sq. ins., an engine with 30 000 lbs. on an axle would cause a pressure of 441 lbs. per square inch on the tie, making no allowance for distribution by the rail. According to investigations of the Forestry Department of the Government, the pines and cedars, when saturated with water, as the surface of a tie is likely to be at times, have a resistance to a 3% penetration across the grain, of from 400 to 500 lbs., corresponding to the assumed working limit of compression. As the condition of a full bearing on a new tie is decidedly improbable, it is by no means impossible that in such woods, the initial cutting under the rail may be due largely to the direct compression.

The injury by splitting from seasoning can be reduced by covering with ballast a larger part of the ties subject to such action, and that from respiking by carefully plugging all spike holes. As to the general practice regarding plugging, the author is not qualified to speak. He understands that it is generally neglected, but he knows of one road which supplies the section men with turned plugs of seasoned elm; and the instructions are that every hole shall be carefully plugged, whether the spike is redriven into it or not, unless the tie is so decayed as to necessitate removal within a year. The results are reported as being very satisfactory and the slight expense of providing the proper material for such use fully justified.

To secure the best results with any tie, it is necessary that the tree be cut at the proper time, the wood seasoned before being placed in the track, and the roadbed well drained. The necessity for cutting at a particular time has been attributed to various causes: The greater amount of sap, the greater activity of the sap, the change of position of the sap in the tree and the change in the character of the sap at certain seasons, which were consequently to be avoided for cutting. The latest dictum on the subject, as contained in the 10th Bulletin of the United States Forestry Division, and reprinted in Professor J. B. Johnson's "Materials of Construction," is:

"Wood cut in the fall is more durable than that cut in summer, only because the low temperature of the winter season prevents the attack of the fungi, and the wood is thus given a fair chance to dry. Usually, summer-felled wood, on account of the prevalent high

temperature and exposure to sun, checks more than winter-felled wood; and since all season-checks favor the entrance of both moisture and fungus, they facilitate destruction."

Whatever the reason, there seems to be no question but that in the northern part of the United States, timber should be cut in the late fall and winter. The value of seasoning seems also to be undisputed, but the record of experience on the subject is small. That the roadbed should be well drained, to prevent the decay of the ties, is evident; and that the ballast should not cover the ends of the ties, so as to prevent the ready escape of water and the circulation of air around them, is good practice which is frequently ignored.

The immediate cause of the removal of the average tie is dependent upon the character of the timber used, the climatic conditions, the severity of tie inspection, both when purchasing and after service in the track, the character of the roadbed and the amount of traffic. It may be decay or the mechanical destruction under the rail. If it is decay, it may be the general failure of the stick, or local rot under the rail. In any case, it may be concluded that anything which will postpone the decay of the wood, will add to the life of the tie, for where the fibers are injured, rot will find favorable conditions for development, and where the rot has begun, the fibers are reduced in strength and quickly destroyed. With certain woods durability can be secured by mechanical means for preventing the breaking down of the fibers under the rail, as by tie-plates. This is especially true of cedar, cypress and redwood. Woods which are of such texture that they resist decay and mechanical injury equally well for considerable periods, and have consequently been the most satisfactory timber for ties, are unfortunately few; being the white, burr, chestnut, post and rock oaks. With most of the American woods, however, nothing will materially prolong the life of the tie which does not essentially change the condition of the sap. Such change may in itself be sufficient to make a timber perfectly adapted for use as a tie, or it may still be necessary to protect it from mechanical injury; the determining factors being the character of the timber operated on or the process used.

The oaks, which have been in the past and still are in some localities the principal source of the best ties, are rapidly being exhausted. It is an especially favored road which can to-day secure any considerable

supply of good white or burr oak from its own territory, and even then it is the policy to use the oak for but a part of the supply, supplementing it with other and less desirable materials. The report of one of the committees of the Roadmasters' Association, made to the annual meeting held in Denver in September, 1898, may be quoted as indicating the general conviction that the day of the hardwood tie is nearing its close. This report recommended:

"That tie-plates with soft ties be used in preference to hard ties without any plate, when the natural life of the soft tie is equal to or greater than the natural life of the hard tie, cost being equal."

The cedar is being more largely used with and without tie-plates, but any largely increased demand for cedar, due to its adoption generally, would probably cause a rapid increase in cost. It is not a wood which is generally or uniformly distributed throughout the country; and even in Chicago, closely adjacent to the cedar supplies, the increased demand of the last 3 years has advanced the price from 25 cents to 38 cents.

The future supply of ties must undoubtedly come from what are now considered the second-class or even waste woods. These, after chemical treatment, will resist decay for from 10 to 12 years and will generally cost less than the white or burr oaks.

One of the causes which has prevented the more general adoption of preservative methods, has been the low cost of timber, even of the best quality. Notwithstanding the generally admitted fact that the supply of such timber is materially less than it was 10 years ago, in many localities the cost is no greater than it was then; and in some cases it is even less. One of the Western roads, which 10 years ago paid 50 cents each for the white oak ties purchased along its own lines, is now paying for the same tie 45 cents; and, owing to stricter and more thorough inspection, it is getting better timber. However, although this road runs through a magnificent timber country, and purchases all the oak offered for sale on the line, it has been found necessary to supplement the supply by importations from the South at higher prices, and by the increased use of soft woods. This year a larger percentage of soft woods will be used than has been the custom.

One explanation of this shortening of the supply without a corresponding increase in cost may perhaps be that given in one of the Government reports on the subject:

"The agencies employed in supplying the enormous quantities of ties required by the railway system may determine, to a great extent, the amount and price of material offered in the market. In many parts of the country the cutting and hauling of ties to railway stations is performed mostly by farmers to whom this labor is a source of ready cash and employment of spare time. Owing to this mode of supply, an abundance of material is steadily offered, giving railways full control in the selection of the best quality and in the price paid; while by a regulation of the rates of freight they can prevent an undue export to the treeless sections of the country, or check materially the lumber industry of certain localities. In this way prices are kept down and a show of seeming abundance is maintained which does not in the least prove the supply to be adequate, but merely shows that the great number of people engaged in this industry are willing to haul ties to the railway over distances steadily increasing as the timber nearest the lines is cut off. Immense quantities of tie-timber are thus cut down and delivered to the railways at prices below its real value. At the time the leading lines west of the Missouri River were constructed, extensive tracts of white oak timber were laid waste in the Western States. And the same process is going on throughout the white cedar regions of the North, from which immense quantities of ties are thrown on the timber market. These channels of supply, based solely on considerations of monetary profit, will continue to bring to market the forest wealth of many sections of the land, but in many instances and at a time not far distant Nature's bountiful supply will be exhausted, and the reckless system of forest clearing will of necessity be a thing of the past."

This was written 12 years ago and the truth of it is evidenced by the greater scarcity, and, so far as many roads are concerned, practical exhaustion, of the white oak supplies in the northern part of the country.

The constant extensions of railroads into new territory, opening up forests before inaccessible, has had even more to do with the stationary prices. A recent number of a trade publication* says:

"So long as railroad rails rest on wood, so long the white oak tie will be wanted above any other available wood. A large proportion of the mileage of western roads has for many years been laid with this tie and on ties brought from the woods of the three white pine States. But the time has now come when this source of supply is failing rapidly.

"The States of Michigan, Wisconsin and Minnesota to-day furnish probably one white oak tie where 5 years ago they furnished five. With the present rate of cutting these ties in these States, the supply 5

* *The Mississippi Valley Lumberman.*

years hence promises to be insignificant. Four-fifths of the white oak ties now being bought by the roads running through these States are brought from the South. In time the supply will be almost wholly from the South.

"Though the supply of white oak suitable for ties has grown so small in the North, the market value of this tie has remained almost stationary because of the recent influx of the Southern tie. The Omaha, the Soo, the Wisconsin Central and the Milwaukee lines have this year drawn the bulk of their white oak from the South. A reduction in freight rates is in part responsible for this."

Experience with Southern oaks when brought North seems to be somewhat contradictory, but is probably unsatisfactory, while the cost is high. As a consequence, it is believed that the supply of ties generally and very shortly will have to come from the inferior grades of timber, with a corresponding large increase in the number of annual tie renewals, unless it is possible to so treat them at a low cost as to give them a life equal or superior to that now secured from the oaks.

Résumé of Report of 1885.

In 1885 a Committee of this Society, consisting of nine members, presented an exhaustive report on the subject of the preservation of timber; which, with the discussion thereon, participated in by sixteen other members, still stands as the authoritative statement on the subject; and many of the general conclusions of the Committee remain uncontroverted to-day. Very naturally, however, during the past 12 years much has been learned, some of the ideas formerly held have been modified and confirmatory evidence produced as to others.

The conclusions in the report on the preservation of ties were:

"If the exposure is to be that of a railroad tie, creosoting is, doubtless, the most perfect process to use, but in view of the expense, it may be preferable to use a cheaper process, dependent somewhat upon the location.

"With our present knowledge, and as a result of this investigation, we believe Burnettizing (the zinc-chloride process) is the advisable process to use for ties at present in this country."

The facts upon which the conclusion as to the sufficiency of Burnettizing was based were the following:

A street railway in Cambridge, Mass., was laid with spruce stringers and sleepers in 1855. In 1883, 28 years later, the stringers

were all worn out, but the president reported that "many, and, I think, the majority of the sleepers are in good condition to-day."

On the Vermont Central Railroad ties were treated in 1856 to 1860, at which time the plant was abandoned and the entire matter lost sight of until 1879, "when an old side track was removed, which had not been in use for several years, and which was nearly covered with earth and grass; still the hemlock ties were then found to be nearly sound" after over 20 years.

"The reasons for abandoning the Burnettizing works upon this road would seem to be that the officers in charge at that time lost faith in the theory, and, as it was an experiment, they did not learn of its value until recently discovered in the manner referred to."

In a wharf and in a street railway in Boston, Mass., spruce plank and track stringers which were Burnettized in 1857 were sound in 1872, or 15 years afterward.

On the Chicago, Rock Island and Pacific Railroad eight spans of Howe-truss bridges built in 1860 were still in use and in fair condition in 1882. On this road 2 000 ties of hemlock, pine, tamarack and cedar were laid in 1866. In 1882 about 75% of the hemlock was still in the track and good for several years longer; the pine and cedar ties had all been removed sometime during the 15 years—the tamarack lasting about as long as the hemlock.

On the Boston and Albany Railroad a 100-ft. bridge, built in Boston by the late E. S. Philbrick, M. Am. Soc. C. E., was provided with green spruce Burnettized ties in 1860 which lasted 9 years. They were replaced by untreated pine, lasting 6 years, and these were replaced by untreated chestnut, lasting 7 years.

Untreated spruce is stated by Mr. Philbrick to have a life of 4 to 5 years.

On the Lehigh and Susquehanna Railroad, L. L. Buck, M. Am. Soc. C. E., reported in 1883 that he had examined Burnettized maple, beech and hemlock ties laid in 1867–68 which had "resisted decay almost perfectly. Most of the treated ties appeared good for 7 or 8 years longer."

This represents the American experience in 1885 favorable to zinc chloride as a tie preservative. There were, in addition, the results of 30 years of German experience, which can be summarized from Councillor Funk's paper in the report as follows: In 1880, out of 60 000 000

ties laid on the German and Austro-Hungarian railroads, about 25 000 000 were impregnated, the majority with zinc.

"As the experience of those railroads that have for from 25 to 26 years impregnated their ties with chloride of zinc under pressure, after steaming and abstracting the sap, has been very satisfactory, and as this system costs only one-half to one-third as much as impregnation with creosote or corrosive sublimate, the majority of roads have adopted the chloride of zinc process.

"The average life of ties impregnated in a rational manner with creosote or chloride of zinc under a powerful pressure reaches:

"For oak ties	19.5 years.
" fir ties	14 to 16 years.
" pine ties	8 " 10 "
" beech ties	15 " 18 "

"The results of impregnation of chloride of zinc and creosote are about equal. But, as the impregnation with creosote costs about three times as much as with chloride of zinc, a majority of the German railroads have gone over to the latter."

In addition to this favorable evidence, there was some which was unfavorable. In 1861 the Erie Railroad built a plant at Owego, N. Y., in which plank and bridge timbers, as well as a few experimental ties, were treated. The results were unsatisfactory, the large timbers, in 8 or 9 years, having decayed at the center, leaving a hard crust on the outside. In the opinion of the Committee, the reasons for the unfavorable results were: The original imperfections in the "Bethel process," it being attempted to extract the moisture (before injection) by the vacuum alone, without preliminary steaming; the selection of bridge timbers which are less adapted to the treatment than ties; operating on entirely unseasoned timber, and, perhaps, the use of insufficient pressure. Referring to the use of green timber:

"As a confirmation of this view, it may be mentioned that, while the bridge timber did not last well, several sets of switch ties, which had been cut about eight months and seasoned, were Burnettized at Owego for the road department of the Erie Railway, and laid in its tracks, with the result that 9 years afterward they did not show a particle of decay. These ties have been lost sight of since, but your committee has in its possession an oak tie, one of a lot of two car loads, seasoned and Burnettized at Owego, which lay in the track at Susquehanna Station for 17 years, and is almost perfectly sound to-day."

The Philadelphia, Wilmington and Baltimore Railroad and the Philadelphia and Reading Railroad built works in 1863 and 1867. The

results were satisfactory as regards decay, but the strength of the timber was so injured that the ties broke in the track.

"The explanation is that the solution was too strong, and the surplus zinc probably crystallized in the sap ducts of the timber, burst some of them asunder, and so made the wood brittle. On the Philadelphia, Wilmington and Baltimore Railroad a solution was used probably 5 or 6% strong, and on the Philadelphia and Reading Railroad the solution was 3¾% strong, while experience in Germany has thoroughly established the fact that for railroad ties the solution should not be over 2% strong, and 1.91% is considered the standard."

The present German practice, as to strength of solution, will be given later. There is no evidence known to the author to confirm the supposition that the surplus zinc contained in solutions of the strength mentioned has ever crystallized in the ducts and injured the fiber. It is probably simply a conclusion based upon an idea.

In the consideration of this record in the United States it is necessary to remember that while the use of zinc chloride as a preservative of wood was patented in 1838, the Bethel process of securing the saturation of the timber by pressure preceeded by partial vacuum was invented in 1840; and an attempt to apply the treatment was made here as early as 1850. The work for the next 20 years was largely experimental. Everything had to be learned as to the proper methods of treatment, the process perfected, and the best materials to work upon ascertained. Timber was then so cheap and plentiful in the United States as to leave but small incentive to devising means of economizing it. That the results of treatment were so largely favorable and the probable causes of failure so evident, is remarkable. The record of the hemlock and tamarack ties on the Rock Island road is difficult to account for, in the light of later experience. After 15 years' service, 75% were reported as still in the track. As will be seen later, the more recent hemlock ties on this road show a shorter average life, and it must be that the previous statement was in error, or the treatment the ties received before and after they were put into the track was more favorable than that of later years.

An endeavor has been made to learn the facts as to the method of treatment followed with these ties, but the only person accessible who had personal relations with the work in 1866 disclaims all recollection of the details and no written records were kept. The author has been told, however, that these ties were cut in December, cross-piled to

season until the following October, then treated and allowed to dry before they were placed in the track. If this is true, their record is probably fully accounted for, and can be duplicated if the same conditions are supplied.

Councillor Funk's statement that the results of impregnation with creosote and with chloride of zinc were about equal, is evidently in error. In the "Organ of the Progress of Railroads," series 1897, published in Wiesbaden, there is a table in which the average duration of various ties on the German railroads is given for the zinc treatment and also with tar oil (creosoted). These are:

Oak ties,	treated with	zinc chloride,	15	years; with	tar oil	24	years.
Beech	"	"	9	"	"	30	"
Pine	"	"	12	"	"	20	"

Progress Since 1885.

At the time of the report to the Society, three plants treating timber with zinc chloride were in operation in the United States, each working with a different process; the difference being in the methods adopted to prevent the impairment of the preservative treatment by the washing out of the zinc salt, which is quite soluble in water. This solubility of the chloride is the only objection to its use, but the exact value of this point is not determined to the author's satisfaction. The evidence already adduced relates to the Burnettizing process proper, in which the zinc chloride alone is injected, with no attempt at any artificial method of preventing its being washed out. However, various methods have been devised to eliminate any objection on this score, and each of the plants in operation in 1885 was working on patented ideas, with this object in view.

The Thilmany Process.

The Thilmany process consisted of the injection of sulphate of zinc or copper, followed by a second injection of chloride of barium, when a chemical change was supposed to take place in the wood ducts, the chloride of barium being changed into an insoluble salt, sulphate of baryta; its chlorine forming chloride of zinc or copper. The insoluble salt was to prevent the soluble one from being washed out. Timber was treated in this way at plants in Defiance, Ohio, and Milwaukee, Wis., and some favorable results were reported. Ties had been treated

for the Chicago and Alton Railroad in 1880, and for the Erie Railroad in 1882. The Government report of Forestry (1884) states that 200 000 soft wood ties treated by this process had been put into the tracks of the Lake Shore and Michigan Southern Railroad. Some were also treated for the Wabash Railroad.

Correct records of the ties on none of these roads are obtainable. Mr. H. C. Draper, Chief Engineer of the Chicago and Alton Railroad, recollects having made an examination of the results on his road which was unfavorable to the ties. Those treated had been of various soft woods growing along the line, and the experiments were not considered successful. No definite information as to the life of these ties can be secured.

On the Erie and the Wabash railroads the results were unsatisfactory, as evidenced by letters of the late Charles Latimer, M. Am. Soc. C. E., and W. S. Lincoln, M. Am. Soc. C. E., the Chief Engineers of these roads. It is supposed that the failures were, in part, due to the expected chemical action not taking place when the substances were confined to such small spaces as the ducts of timber.

The Zinc-Gypsum Process.

The zinc-gypsum process consisted of one injection of a solution of chloride of zinc and gypsum. It was claimed that the gypsum crystallized in the ducts of the wood, thus stopping them up and preventing the zinc from being washed out. As the process was then new, no evidence was submitted in the report, except statements of chemists showing the apparent impossibility of washing the zinc out in laboratory tests. A plant was operated at St. Louis for several years, but was not a success financially; it was dismantled, and efforts to secure any facts as to the value of the work done have been unsuccessful.

The Wellhouse or Zinc-Tannin Process.

The Wellhouse or zinc-tannin process consisted of the injection of a solution of zinc chloride containing a small amount of glue, followed by another injection of a solution of tannin, the effect of the tannin, it was claimed, being to form with the glue small particles of artificial leather, insoluble in water, which would plug up the ducts and retain the zinc chloride. A plant was in operation in St. Louis under the management of the late Joseph P. Card, Assoc. Am. Soc. C. E.

TABLE No. 1.—Examination of Ties on the Atchison, Topeka and Santa Fé Railroad.

At Topeka.		July 2d, 1891.			
Kind of wood.	No. of ties.	Date laid.	No. now in track.	Percentage.	Years in track.
Colorado pine	49	July 7th, 1881	Destroyed by wreck.		
Cottonwood	87	August 1st, 1882	43	49.4	8.9
Sweet gum	51	" "	10	19.6	8.9
Red oak	48	August 18th, 1882	29	60.4	8.8

At La Junta.			July 6th, 1891.			May 11th, 1893.			March 14th, 1897.		
Kind of wood.	No. of ties.	Date laid.	No. now in track.	Percentage.	Years laid.	No. now in track.	Percentage.	Years laid.	No. now in track.	Percentage.	Years laid.
Black oak	49	May, 1882	49	100	9.1	47	96	11	44	90	14.8
White cottonwood	49	" "	47	96	9.1	11	22.4	11	8	16	14.8
Cottonwood	50	" "	46	92	9.1	22	44	11	17	34	14.8
Colorado pine	50	July, 1881	50	100	10.0	20	40	11.8	None.	.....	.....

The first timber was treated in 1879. In the following years paving blocks, experimental lots of ties for various railroads and miscellaneous timber were treated. The ties were laid in the vicinity of St. Louis and on the Atchison, Topeka and Santa Fé Railroad. Regarding this work O. Chanute, M. Am. Soc. C. E., says:

"In 1885 I went to St. Louis upon the occasion of the repairing of the St. Louis Bridge. This had been laid in 1879 with gum-wood blocks prepared by the Wellhouse process. These blocks, originally 5 inches thick, had worn down so thin as to give trouble, but they were sound after 6 years' exposure. Less than a dozen were found decayed in the whole length of the bridge. The Burnettized pine stringers were also sound, and the local railroads reported ties treated by the Wellhouse process to be a success, except one lot laid in mine screenings and 'bone' at East St. Louis."

Of the experimental ties put into the tracks of the Atchison, Topeka and Santa Fé Railroad in 1881 and 1882, examinations were made by Mr. Chanute in July, 1891; of those at La Junta, Colo., and Topeka, Kans., and in May, 1893, and March, 1897, at La Junta. James Dun, M. Am. Soc. C. E., kindly had examinations made in both places in December, 1898. He found some of the treated ties still in

the track, but the tin tags identifying each tie had entirely disappeared, or had become so badly scarred that no count of any value could be made. The records of the previous examinations are given in Table No. 1.

Atchison, Topeka and Santa Fé Railroad Works.

The first road to adopt the use of the treated ties, other than as an experiment, was the Atchison, Topeka and Santa Fé Railroad, of which A. A. Robinson, M. Am. Soc. C. E., was then Chief Engineer. This road, in 1885, had a plant built, under Mr. J. P. Card's direction, at Las Vegas, N. Mex., consisting of two retorts 6 ft. in diameter and 106 ft. long, which operated on mountain pine. The Wellhouse or zinc-tannin process was used except from 1889 to 1893, when the Burnettizing (proper) method was followed. The number of ties treated was as follows:

1885	111 503	1892	187 270
1886	333 298	1893	178 167
1887	189 386	1894	247 566
1888	285 104	1895	256 887
1889	169 174	1896	336 070
1890	105 753	1897	380 523
1891	128 568		
Total	2 909 269		

These ties have been used from Dodge City, Kans., to El Paso, Tex., and Silver City, N. Mex.; and a few in Eastern Kansas and Oklahoma Territory. The majority, however, have been used in New Mexico. The life of the untreated mountain pine is given as not to exceed 4 years. Complete records of the treated ties are unobtainable, but through the kindness of Mr. James Dun, Chief Engineer; Mr. H. U. Mudge, General Superintendent, and Mr. George W. Noyes, Manager of the Las Vegas works, the following facts have been secured:

"The New Mexico Division was built in 1880, with native untreated ties. We commenced renewals in 1885, with native pine treated ties, and since that time no other class of ties has been used. The untreated ties were, of course, all removed prior to 1893, so that ties taken out since that time are evidently all treated ties.

"We have assumed that after 100% of the original ties have been removed, all ties taken out were then treated ties. For the 5 years, 1893 to 1897, inclusive, we inserted on the New Mexico Division, on 380 miles, 414 189 treated ties, an average of about 217 ties per mile per year. The renewals for this period were about normal, the ties in this track being in very good condition; therefore, the average life of these ties can be put down as 11 or 12 years.

"As to peculiarities in the length of life of ties due to the character of the soil, I have to say that in New Mexico, ties last much longer in rock ballast than in soil, and decay very much faster in sandy soil than in clay. There are still (July, 1898) a great many treated ties in track branded 1885. You will also understand that a good many of the ties removed are taken from sharp curves and were removed on account of being spike killed.

"On the New Mexico Division we are using treated native pine ties. On other divisions we have been using cedar and oak; usually oak in main lines only. The life of oak ties in heavy main line service is about 6 years; of cedar ties in light branch service, about 10 years.

"Relative to the Burnettized ties laid in 1889 to 1893, we have as yet been unable to demonstrate whether the life of these ties is less than that of the Wellhouse process, as ties laid in 1889 have not yet shown signs of deterioration."

Beginning in 1897, the records of the ties on the New Mexico Division have been more carefully kept, and for future years they will show the number of removals and the causes. Mr. J. E. Hurley, Division Superintendent, has furnished a statement (Table No. 2) covering the years 1897 and 1898.

TABLE No. 2.—Treated Ties Removed on the New Mexico Division of the Atchison, Topeka and Santa Fé Railroad in the Last 9 Months of 1897 and in 1898.

Year removed.	Years of Treatment.												Total removed.	Average life.
	1885.	1886.	1887.	1888.	1889.	1890.	1891.	1892.	1893.	1894.	1895.	1896.		
1897...	13 114	17 540	15 527	10 327	2 459	2 552	566	560	55	104	17	10	62 831	$10\frac{1}{2}$ yrs.
1898...	14 580	17 239	24 662	18 424	5 575	3 649	1 917	5 444	1 900	274	118	49	93 822	$10\frac{1}{2}$ "
	27 694	34 770	40 189	28 751	8 034	6 201	2 483	6 004	1 955	378	135	59	.....	.....

"I am not able to give you the exact number removed from different causes, but would say that 1% of the total will cover all ties removed on account of being broken by wrecks and causes of a like

nature, the balance being removed on account of being rotten, worn out," etc.

The average life of the ties removed does not quite equal that estimated for them by Mr. Mudge, but it does not follow that his estimate is in error; although his premises apparently are. With the renewal of 62 831 treated ties in 9 months of 1897, and of 93 822 in 1898 it is improbable that the 414 189 ties renewed in the 5 years, 1893 to 1897 inclusive, could all have been treated ties. It will be noted that the average life of the ties removed in 1898 was longer than that of those removed during 1897; and it is understood that the 1885 ties are not yet all removed. It is natural to expect that the life of the ties removed each year will continue to increase for some years to come, certainly until all the 1885 ties are removed. As the best results can hardly be expected from the treatment of the first year, it is very probable that 12 years' life will be reached by these ties.

In the Las Vegas works, ties are steamed from 2 to 6 hours, depending upon their condition when received, green timber requiring longer time. They are in a vacuum for $\frac{1}{2}$ to 1 hour, and under pressure for $2\frac{1}{2}$ to 3 hours. Prior to 1890 the chloride was prepared at the works, but since then it has been purchased, in a fused condition. The purity has varied from 95 to 97 per cent. "The amount of zinc chloride injected varies in different years, varying from 0.28 to 0.47 lbs. of chloride per cubic foot of timber treated, which includes sawn lumber of all kinds and piling." Seasoned timber is preferred; and after treatment ties are at once shipped to their destination. They are then supposed to be unloaded, piled up in lots of ten, in triangular piles, so arranged that only one end of any tie is on the ground; and allowed to season for about 30 days. In point of fact, they are generally put into the ground immediately, full of water; on account of the difficulty in providing for the demands of the road with the present facilities. In Mr. Noyes' opinion, to secure the best results, ties should be seasoned for 6 months before treatment and for the same time thereafter.

In 1896 this plant was enlarged by the addition of a third retort. In 1897 a large plant, consisting of four retorts, was constructed by private parties, at Somerville, Tex., where it is expected to treat large quantities of ties for the Santa Fé system; and this railroad has just completed another plant at Bellemont, Ariz., consisting of two

retorts, where ties will be treated for that part of the system. The officials of the road express themselves as satisfied with the value of the treatment, but are discussing the relative values of the zinc-tannin and the plain zinc-chloride methods. The additional cost to inject the glue and tannin amounts to several cents per tie; and they are in doubt from their own experience whether the increased cost results in an increased life, and, if so, to what extent.

Union Pacific Railroad Works.

The Union Pacific Railroad had some experience with wood preservation in 1867 and 1868 when works were erected at Omaha and operated on cottonwood ties for 1 year, when they were discontinued as it was impracticable with the plant at hand to prepare more than one-tenth of the ties required.

In 1886 new works, consisting of two retorts, were erected at Laramie, Wyo., under Mr. Chanute's direction. They were operated for 2 years by the railroad, and 242 000 ties were treated, when work was suspended for reasons of temporary economy. It was also reported to have been because the United States Government stopped the cutting of ties on its lands, and because of prejudice on the part of some one in authority. Whatever the reason, the works were shut up, were partially burned in 1889, and were then dismantled.

While operated, the Wellhouse process was used. According to the detailed record of the operation of this plant from July 26th to August 21st, 1886, the strength of the solution of chloride corresponded to 1.90 on the Beaumé scale, representing 1.46% of chloride in the solution, and the absorption of chloride was 0.65% of the weight of the tie, or 0.78 lb. per tie, the timber (mountain pine) weighing 29.51 lbs. per cubic foot.

Mr. Wm. Ashton, Division Engineer of the road, in a letter to Mr. Chanute, dated January 31st, 1898, speaking of the preserved ties, says :

" This work was done prior to my assignment to this portion of the line, consequently I have not had the opportunity to watch the service given by these ties fully as I would have liked. For the past 3 or 4 years I have been observing what remained of them and have talked frequently with our roadmasters, one of whom I understand has been on the portion of the line on which these ties were laid since they were placed in the track. From what I know and have been able to gather concerning these ties would state as follows:

"Ties treated were native pine and spruce, Oregon fir, elm, ash, maple and oak, by far the larger number being of native pine and spruce.

"The native pine and spruce ties began to fail in 1893; those removed during this year seemed to be mostly 'top-cut' ties. In 1894 most of the top-cut and middle-cut ties were taken out, while in 1895 and 1896 practically all the pine and spruce ties in the main track were removed. These ties were not cut by the base of the rail as badly as untreated ties of the same timber, and their failure could seldom be attributed to this cause. It was observed that the heart and bottom of these ties failed first, leaving a hard shell-like top. There are still a number of these ties in sidings.

"The Oregon fir ties were usually small, 7 x 8 ins., and were found to split badly in spiking. They failed a year or two sooner than the native pine and spruce ties.

"The elm, ash and maple ties have all been taken out during the past 4 years. The service of these ties would have seemed to be about the same as those of native pine and spruce.

"A few of the oak ties are still in the main track. We find they are getting very brittle and are breaking badly. They will probably all be taken out during the years 1898 and 1899."

The above report indicates an average life, for the mountain pines and spruces in the main track, of about 9 years. In January, 1896, Mr. Ashton read a paper before the Denver Society of Civil Engineers, on the "Average Life of Cross-Ties on Steam Railroads in Colorado."*

In this paper he says :

"Under ordinary circumstances my experience on the lines in Colorado has been that ties of Colorado pine, balsam and white spruce will last in track 3½ to 4 years under ordinarily heavy traffic. Under light traffic on a well-tied track, they will probably average 4 years. On our main line we usually take out about half of these ties after 3 years' service, and the balance the fourth year."

This would indicate the result of treatment on these ties to be an unqualified success, although the life attained is materially shorter than that reported for the Atchison, Topeka and Santa Fé Railroad, with practically the same material, but perhaps a drier exposure.

At the author's request, Mr. Berry, the present Chief Engineer, had an analysis made of the chloride of zinc remaining in one of the mountain pine ties taken out of the track in 1898, after 11 or 12 years' service. The analysis showed 0.146% by weight of the chloride, the amount being the same in samples from the center and from near

* *Journal* of the Association of Engineering Societies, Sept., 1897.

the outside of the tie. In the report of Mr. W. L. Robinson, chemist of the road, he says: "The ties, when first taken from the works, seldom carried over 0.25% of metallic zinc" (or 0.52% of zinc chloride). "The ties (analyzed) are in a very good state of preservation."

Works at Chicago, Ill.

This plant is the successor of the St. Louis plant, in which the Wellhouse process was originally worked by Mr. J. P. Card, and was built in 1886 by Mr. Card and Mr. Chanute, operating under the name of The Chicago Tie Preserving Company. It consisted first of two retorts, to which a third was added in 1891 and a fourth in 1894, and has treated, up to December 31st, 1898, 4 007 946 ties for the Chicago, Rock Island and Pacific Railroad, besides small lots for other parties. The usual methods of the Wellhouse process were followed until 1896, when a modification was introduced, consisting of the omission of the glue from the zinc-chloride solution and the subsequent injection of the glue separately, followed by a third injection of the tannin.

This change was based upon experiments, and the idea that the admixture of glue with the chloride solution lessened its fluidity and consequently increased the difficulty of securing the injection of the desired amount of the chloride. It might also be argued that the province of the glue being with the tannin simply to plug up the ducts to prevent the washing out of the zinc, the only proper place for it is at the ends of the ducts and medullary rays; and that it is unwise to introduce into the body of a tie perishable organic material, like glue, to increase the duty of the chloride, and where it is practically impossible to force the tannin to it, by a subsequent injection. This modification, by necessitating three injections in place of two, somewhat increases the expense of the treatment, and while it is undoubtedly an improvement, the period of its use has been too short to demonstrate it, in increased life. The best evidence of its value is in the increased absorption of chloride obtainable.

The ties treated for the Rock Island road have been entirely hemlock and tamarack. The number by years is given in Table No. 4. The number of tamaracks being relatively small, no distinction between them is made in the following, except where the tamaracks are specifically referred to.

It is extremely unfortunate that the records of service of these ties are incomplete and of doubtful value. It is stated that practically none of the ties was removed from the track until 1892. At the time they began to come out, instructions were issued by the management of the road, that all treated ties removed should be piled up and not destroyed until inspected by a representative of the Tie Preserving Company. This was done in 1892 and 1893, except for those removed from a lot of 39 604 shipped west of the Missouri River in 1886; the removals in 1892 being 2 319 or 2.1% of the ties laid 6 years before east of the Missouri River; removals in 1893, 14 282, or 3.8% of those laid 6 and 7 years before, in 1886 and 1887, or 13% of those laid in 1886. The removals for these 2 years, 16 601 ties, represent 15.3% of those laid in 1886 east of the Missouri River, so that at this time there were still in service 84.7% of the ties laid 7 years before.

In the years subsequent to 1893, the ties removed were not inspected and counted by an agent of the Chicago Tie Preserving Company, but were reported by the section foremen to the roadmasters, and by the latter to the Superintendent of Maintenance of Way, who compiled a general statement. The removals, as given for the tracks east of the Missouri River for the years 1894 to 1897, are shown in Table No. 4. From these statements of the annual removals it was computed by the Tie Preserving Company that the average life of the treated ties was $10\frac{1}{4}$ years, an increasing percentage coming out each year, up to the twelfth, when all were supposed to have been removed.

Within the past year doubt has been thrown by some of the railroad officials, upon the accuracy of these yearly returns, and it has been claimed that the ties were averaging less than 7 years. Unfortunately, the ties were not marked in any way to indicate the year treated, until quite recently, and there had been very considerable changes in the divisions of the road, in the sections and in the methods of managing the track system. The records have become inextricably mixed, so that it is impossible to determine when and where the earlier treated ties were laid. This reduced the question of the average life to a matter of recollection and opinion, and while opinion had been favorable up to 1894, it now swung the other way. Finally, in July, 1898, Mr. H. A. Parker, Second Vice-President and Chief Engineer, ordered a count by sections to be made on August 1st, to determine how many

ties of each kind (oak, cedar and treated hemlock) were then in the tracks of the road, each section foreman counting the ties on his own section.

The result of this count is shown in Table No. 3.

TABLE No. 3.—Ties in Tracks of Chicago, Rock Island and Pacific Railroad, August 1st, 1898.

Kind of tie.	White oak.	Cedar.	Treated hemlock.	Pine.	Treated pine.	Totals.
In main track	4 541 235	3 308 675	2 545 240	29 249	93 940	10 518 339
" side "	484 626	877 143	345 948	4 586	10 010	1 722 313
Totals	5 025 861	4 185 818	2 891 188	33 835	103 950	12 240 652
Percentage in sidings	9.7	21	12	13.5	9.7	14.1

The totals in Table No. 3 represent averages of 2 430 ties per mile in sidings, 3 050 ties per mile in main track and 2 950 ties per mile in all tracks. Mr. Parker is satisfied that this total number of all ties is substantially correct. The treated pine ties shown in the table were treated at works in Texas.

It remains to deduce from these figures the probable life of the treated ties. Up to the same date (August 1st, 1898) there had been treated, hemlock and tamarack ties to the number of	3 860 295
Less, shipped to the Rock Island and Peoria Railroad and not in above count	72 194
Total to account for	3 788 101
" counted in track	2 891 188
Balance presumably renewed	896 913
The treated ties were at that time being shipped west of the Missouri River, a distance of from 500 to 1 000 miles, and many were in transit, there having been shipped from the treating works during the last ten days of July, 29 209 ties; and there were also, certainly, some distributed but not in track. Assuming as fair to deduct for these two items	50 000
Leaves, as the probable renewals	846 913

This absorbs all the ties treated up to 9 years previous, or to the close of the treating season of 1889, amounting to 825 796, of which 21 623 had been shipped to the Rock Island and Peoria Railroad, so that the tracks under discussion received 804 173.

Leaving, presumably, to come from the ties treated in the following year, 1890, 42 740.

This number is 17% of the ties treated in 1890 and is to be deducted from the 9 years, leaving 8.83 years as the probable average life of these treated ties, provided there was no material error in the general count made in August.

The author does not consider this count conclusive as to the life of the treated ties on this road. The time thus determined is shorter than indicated by the removals during 1892 and 1893, in the latter year there being still in the track 85% of the ties laid 7 years before; and as indicated by ties laid on four sections of the same road west of the Missouri River; as well as by those tested on the Pennsylvania Railroad and the Duluth and Iron Range Railroad, as described later. It is extremely difficult to distinguish the treated hemlock and tamarack from cedar, by surface inspection in the track, if indeed it is possible to do so in many cases without cutting the tie, and it seems quite possible that many treated ties may have been reported as cedar; but, in the author's opinion, based on personal experience with the ties, the mistaking of cedars for hemlocks and tamaracks would not be as likely to occur. The only reason given for doubting the annual reports was that they were based on the reports of the section foremen, and had been made in the regular course of work, without any especial effort to secure accuracy. Since the August count, however, orders have been issued to keep these records accurately, and this paper has been held open until the reports of the renewals for the year 1898 became accessible, in order that the probable value of the previous year's reports could be gauged by this one. Table No. 4 gives this as well as all other data that can be secured. It will be noted that the total renewals on the lines east of the Missouri River are 481 947. The total number of treated ties put into the same tracks for the years 1886, 1887 and half of 1888 is 461 765; so that, if the annual reports of ties removed are correct, all ties put in since about the middle of 1888 are still represented in service, and the average life of over 10 years, as figured by the Chicago Tie Preserving Company,

is correct. Since the above was written it has been discovered that the treated tamaracks in the tracks west of the river have been confounded with the treated pines and reported as such—to what extent is unknown. While the number so omitted from the count of the "hemlocks" cannot be very large, it may be 50 000, and the life of the treated ties would be correspondingly increased.

All these figures, based on removals without any knowledge of the year of treatment of the particular ties removed, are extremely unsatisfactory and unscientific; but they are the best obtainable and are so presented.

TABLE No. 4.—TREATED TIE RECORD, CHICAGO, ROCK ISLAND AND PACIFIC RAILROAD.

Year.	No. treated.	Shipped west of Missouri River.	Shipped to R. I. & P. R. R.	Remainder charged to track east of Missouri River.	Removals from tracks east of Missouri River.
1886.....	148 816	39 604	8 048	101 164	.....
1887.....	265 377	.....	3 060	262 317	.....
1888.....	201 272	.....	4 703	196 569	.....
1889.....	210 331	.....	5 812	204 519	.....
1890.....	253 245	.....	27 271	225 974	.....
1891.....	339 423	981	23 300	315 142	.....
1892.....	325 736	15 123	.....	310 613	2 319
1893.....	405 267	100 000 (?)	.....	305 267	14 282
1894.....	430 302	110 048	.....	320 314	33 344
1895.....	368 630	225 669	.....	142 961	62 245
1896.....	297 252	213 613	.....	83 639	71 695
1897.....	425 459	324 876	.....	100 583	137 708
1898.....	336 776	283 612	.....	53 164	160 354
Total.....	4 007 946	1 313 526	72 194	2 622 226	481 947

In 1886 a large number of treated hemlocks and untreated white oaks was laid in the tracks on Sections 1, 2, 5 and 9, west of the Missouri River. The renewals on these were reported in the years 1891, 1892 and 1893.

Inquiries in 1898 showed that no later separate accounts had been kept, but the Roadmaster, Mr. J. D. Sullivan, in September of that year, had a careful count made to determine the number then in the track. These various figures are given in Table No. 5. It will be noted that on Sections 1 and 2, 28.9%, and on Sections 5 and 9, 58.8%, of the treated hemlocks were still in service after 12 years. Mr. Sullivan adds "the ties still in the track are in fair state of preservation, and the best will last 4 years more." The record of the oaks is also given in Table No. 5.

TABLE No. 5.

Section.	No. of ties.	Renewed in 1891.		Renewed in 1892.		Renewed in 1893.		No. in Track Sept., 1898.		Percentage removed.	
		No.	Percentage.	No.	Percentage.	No.	Percentage.	No.	Percentage.		
					Treated Hemlock.						
1....	8 000	0	0	0	0	200	2.5	1 488	18.6	81.4	Average, 71.1
2....	8 000	0	0	0	0	425	5.3	3 137	39.2	60.8	
5....	3 050	0	0	0	0	67	2.2	1 963	64.4	35.6	Average, 41.2
9....	2 800	0	0	0	0	0	0	1 477	52.7	47.2	
	21 850	0	0	0	0	692	3.2	8 065	36.9	63.1	
					Natural White Oak.						
1....	10 000	0	0	487	4.9	800	8.0	3 184	31.8	68.2	Average, 74.1
2....	10 000	418	4.2	658	6.6	1 120	11.2	1 983	19.8	80.2	
5....	15 000	150	1.0	576	3.8	805	5.4	2 732	18.2	81.8	Average, 74.5
9....	15 200	282	1.8	685	4.5	1 335	8.8	4 979	32.7	67.3	
	50 200	850	1.7	2 406	4.8	4 060	8.1	12 878	25.6	74.4	

Mr. Sullivan states that in 2 years practically all the oaks will have been removed.

On Sections 1 and 2, 71% of the hemlocks and 74% of the oaks had been removed after 12 years and on Sections 5 and 9, 41% of the hemlocks and 74% of the oaks. As to the variation in the life of the hemlock on the different sections, Mr. Sullivan says:

"The cause is the difference in dirt and ballast, Sections 1 and 2 being close to the Missouri River, and dirt, largely silt and sand, with 3 miles of rock ballast in the former. Sections 5 and 9 are 24 and 48 miles west of the river, and the ground is largely black firm prairie loam. All kinds of woods, except cedar, are short-lived in river bottom and bluffs."

If the hemlocks now in the tracks last 4 years longer, for the best, as Mr. Sullivan states, it is fair to put the average remaining life of all the ties now in at 2 years. On this basis the average life of all the treated ties would figure for

Section 1..... 10.27 years.
" 2..... 11.13 "
" 5..... 12.34 "
" 9..... 11.62 "

or for all four sections 11 years. These averages are derived thus, taking that for the four sections as an example:

Renewed in 1893, after 7 years' service	3.16	×	7	= 22.12
" 1894, 1895, 1896, 1897 and 1898 (average $9\frac{1}{2}$ years)	59.94	×	$9\frac{1}{2}$	= 569.43
Remaining in track now, good for 2 years more	36.9	×	14	= 516.60
	100.00	×	11.08	= 1108.15

Average, 11.08 years.

The life of the treated ties on this road has undoubtedly been less than is readily obtainable by a more rational practice than was the custom prior to 1895. Until then the Tie Preserving Company treated all ties sent to it, regardless of their condition as to moisture or soundness. Since 1895 this has been changed, and the ties are re-inspected, when received at the works, by the Preserving Company's men, and if not sound are condemned and turned back to the tie contractor; and if the ties are too wet for treatment, they are piled up until sufficiently dry to take the proper amount of chloride.

In 1890 the Chicago Tie Preserving Company treated some experimental ties for the Duluth and Iron Range Railroad. These consisted of 85 white pines, 85 tamaracks, and 86 Norway pines. They were cut during the winter of 1889–90, treated in October, 1890, and placed in track very soon after, together with 10 Norway pine, 10 white pine and 10 tamarack untreated ties, which were cut at the same time and off the same land as those subjected to treatment.

"The roadbed in which these ties were laid is in glacial drift and is what you might call gravelly hard-pan, carrying some lime, and the drainage is good; the ballast is a fairly clean and reasonably coarse gravel."

Mr. J. L. Greatsinger, President and General Manager, had an examination of these ties made in February, 1898, and wrote Mr. Chanute as follows:

"In the winter of 1895 and 1896, we were obliged to tear up the track with dynamite where these ties were originally laid. In doing so, very few of them were broken, but nearly all of them went through the ordeal remarkably well and were relaid in double track in the spring of 1896, as the grade at this point was lowered about 8 ft. At this time an inspection was made of them, and it was decided to lay part of them the same side up as before, while a few of each kind were reversed; and as the same rails, 80 lbs., were used in relaying, it did

not necessitate, except in a few instances, making new spike holes, and where this was done the old holes were plugged.

"While we have but 5 or 6 ins. of snow, yet it is difficult to give as close a report on their condition at this time as it will be next spring, but I am now able to report as follows:

"*White Pine (One Notch).*—The rail has cut into the tie but very little, the greatest depth being $\frac{1}{8}$ in. By cutting into the top of one of the ties, the wood appeared apparently as good as new.

"*Tamarack (Two Notches).*—The rail has cut into them a little more than the white pine; about $\frac{1}{4}$ in. All sound, and otherwise appear as good as new.

"*Norway (Three Notches).*—The rail has cut into these about the same as tamarack, a $\frac{1}{4}$-in. cut in the top of the tie. Found the wood in fair condition, but not as good as either tamarack or white pine.

"In general terms, the process of preservation has not only prevented decay, but has made the wood much denser, thus materially reducing the cutting usually done by the base of rail, and judging from the results already attained, these ties are yet good for 15 years, and, had we used tie-plates, for a much longer time, as the destruction of the tie will be from wear and not from decay.

"Next spring I will take out some of each kind and send you some sections of them under the rail bearing and through the spike holes, which will enable you to see the effect.

"I send you by express, some pieces that were cut from the ties the first of this month, for your inspection. I consider this a good showing, as our service on ties is the most severe of any I know of in this country, as there have been times when we have averaged for every day in the month over 100 trains passing over them every 24 hours. Our heaviest standard locomotive, ready for service, weighs about 120 tons, and our average ore car, loaded, carrying 24 gross tons in a car, itself weighing about 24 000 lbs.; length over all, 24 feet."

In response to an inquiry by the author, Mr. Robert Angst, Chief Engineer, under date of June 22d, 1898, writes:

"The untreated ties you inquire about have reached the end of their career, and are now in the ash heap. They were a fair average of the untreated ties on our line, the effective life being between 7 and 8 years."

In 1891, 200 tamarack and 200 hemlock ties were treated for, and placed in, the tracks of the Pittsburg, Fort Wayne and Chicago Railway. Thomas Rodd, M. Am. Soc. C. E., Chief Engineer, says of them (1898):

"After these ties were put in the track we watched them pretty carefully and for about 3 years they cut rather more than an oak tie.

After that, however, they seemed to cut less than an oak tie, and are to-day in good shape in our main track."

In the spring of 1898 two of these ties had decayed, and were removed. An examination after removal satisfied Mr. Chanute that the two ties were from dead trees, and their failure after 7 years' service he attributes thereto. In 1896 and 1897 the same road put in track about 50 000 hemlock preserved ties; and during 1898 there have been treated for it 13 000 beech and 11 000 black and red oaks.

The Maintenance of Way Committee of the Association of Transportation Officers of the Pennsylvania Lines made a full report on the subject of ties 2 or 3 years ago, in which it was recommended that 100 000 ties, preserved with zinc chloride, should be laid each year on their lines.

There have been a number of ties treated for the Rock Island and Peoria Railroad, a subsidiary line of the Chicago, Rock Island and Pacific Railroad, operated under separate management. An effort was made to secure data from this road as to its experience with these ties, but nothing but very general statements could be had. From the Chicago, Rock Island and Pacific Railroad it was learned that the number of treated ties delivered to the road was as follows:

1886	8 048
1887	3 060
1888	4 703
1889	5 812
1890	27 271
1891	23 300
Total	72 194

In response to an inquiry, Mr. J. E. Breckenridge, Roadmaster, wrote, under date of June 27th, 1898:

"Our ballast at that time" (when the ties were put in) "was mostly gravel, and cinders in places on top of gravel. On prairie soil, gravel ballast, ties lasted about 4 years; on timber soil, gravel ballast, about 7 years; around coal shafts with slack ballast, about 3 years. I think those ties might have done better had they been piled up for one year and got thoroughly dry. The ties seemed to come to us from the vat, and were generally put in track very soon after they were received."

In reply to a later inquiry as to the number and location of any ties still in track, Mr. Breckenridge wrote: "There are none of the ties you speak of in track now."

The author could see no reason to expect any such short life for the ties as this indicated, judging from the experience of others with them; and as inquiries of the higher officials of the road produced similar views to those of Mr. Breckenridge, it was determined to test by actual examination and count, the statement as to the ties being all out; and to determine the reason, if such was the fact. Accordingly, S. M. Rowe, M. Am. Soc. C. E., who was for some time in charge of the Las Vegas treating works, was engaged to walk over the entire line of 118 miles and count all the treated ties he could find. Mr. Rowe made this examination in November and December, 1898, and in his report practically gave the number of ties on each mile separately, so that his count could be readily verified or disproved. He absolutely identified 20 272 treated ties and estimated that there were 2 000 more covered by snow or surplus ballast—practically 30% still in service—indicating how easy it is to lose the reckoning of ties in track. Mr. Rowe states:

"The ballast consists about as follows: From Rock Island yard to Milan, mostly sand, quite fine, taken from local sand pit. The main line eastward as far as Toulon is mainly fine gravel, some places very profuse, covering the ties many places so as to prevent inspection, while much of the distance the ballast is thin.

"Almost all of the coal branch, and that portion of the main line east of Toulon, is ballasted with burnt clay, a very good material for the purpose. The burnt clay ballast is in ample quantities, covering the track entirely for considerable distances and banked against the ends of the ties so as to materially retard drainage.

"In consequence of this, the track is down and ties are churning in the softened track. To what extent this has affected the lasting qualities of the cross-ties, I am not prepared to say, but the matter is here noticed with reference to that question.

"The identification of the treated hemlock ties was not difficult, as the grain of the wood, the presence of bits of the bark and the well-defined shade of the surface of the timber, were all an aid in this; but the most striking characteristic of the hemlock was what I would call its 'integrity'; that appearance of soundness of wood, and especially the absence of material wear under the rail. To fully understand this, a personal examination is necessary, as while the superiority of these treated ties is so marked, the actual condition is hard to put into

words to express it fully. This is especially noticeable between Crampton and Orion on the Cable Branch.

"In regard to the identification of the tamarack it was found very difficult, as the difference of the grain from that of the cedar was less marked, and only the greater firmness of the wood and the shade of color due to the treatment were available, and, in consequence, many were missed in checking.

"Without a more searching examination of these ties yet in track, it is difficult to form an opinion as to the time they will serve. I would judge that perhaps 25% would be removed in a relaying of the track, and that the service of the other 75% would be from 2 to 5 years if properly handled. Very few of the ties (hemlock) were found in the waste piles.

"The resistance to rail wear was remarkable, and when the hemlock was laid with oak, the wear was inappreciable. When laid here and there, interspersed with cedar, the wear was greater, but still insignificant compared to that of the cedar, and this fact aided very much in finding them."

Southern Pacific Railroad.

The profession and the railroads owe a great debt to Mr. J. Kruttschnitt, General Manager of the Southern Pacific Company, for the very careful and complete manner in which he has had kept the records of the ties treated and used on his road. His is the only company which can furnish complete data on the subject; and when the labor entailed is considered, especially after preliminary tests have indicated the success of the treatment, so that there is some confidence in it, it is the more remarkable that he has so consistently and constantly devoted so much of his own time to it, as is necessary to insure its continuance. There can be no question of the value of these records to the railroad, and the slight expense is thoroughly justified, and would be, even if much greater than it is. One of the difficulties in connection with the treatment of ties in the United States in the near future is likely to be this one of proper records. Treated ties, like the untreated ones, will show great, and, to the casual or superficial observer, entirely unaccountable, variations in life; and, unless it is possible to determine definitely by total averages the life secured from the ties, there are apt to be periods when those in authority will conclude—from impressions, not records—that treatment is a failure and a waste of money. Even under favorable conditions, ties will develop eccentricities which will demand investigation

and expert consideration, if the best results are to be obtained. This has been the case already in several instances in the United States and abroad, and as more roads adopt the process and attempt to let it run itself, failures are bound to occur, without the records which, if properly kept, would indicate the cause and permit the remedy to be applied.

All the treated ties on the Southern Pacific Railroad, except the bridge ties, which are creosoted, are preserved by the old original Burnettizing method, viz., the injection of chloride of zinc only, with no other treatment to prevent the leaching out thereof. The use of these ties was commenced on the Texas and New Orleans road, a part of the Southern Pacific, Atlantic System, in 1887, and until 1891 they were treated at the works of the Houston and Texas Central Railroad, at Houston, Texas; after that date, at the plant of the Southern Pacific road, erected near the same place.

The operations on this road are described in a very complete and thoughtful report made to Mr. Kruttschnitt by Mr. J. T. Mahl, Engineer of Maintenance of Way of the Atlantic System, under date of July 9th, 1898. The following data are from this report, supplemented by information in letters by Mr. Kruttschnitt; the quotations are from from the report:

"The timber treated is long-leaf and short-leaf yellow pine from Eastern Texas and Western Louisiana." It is all sap wood, and, untreated, has a life of only $1\frac{1}{2}$ to 3 years, being the cheapest wood obtainable. The heart wood of the same timber, untreated, would have a life of from 6 to 9 years.

"It is felled at all the seasons of the year, and no attempt is made to season it, either before cutting into ties or after receiving it at the works, previous to treatment." However, "in treating the large number of ties we are called on to treat for this company and its affiliated lines, some timber necessarily must be stored at the works, and this receives natural seasoning to some extent. When ties are treated some care is taken in charging the cylinders to fill them as nearly as possible with ties of the same age, so as to secure uniform treatment; for if green ties and ties which had been seasoned for 3 to 6 months were subjected to the same treatment, the percentage of absorption of the seasoned ties would be much greater than the absorption of the green ties."

The strength of the solution of chloride formerly used was $2\frac{1}{2}°$ of the Beaumé scale, corresponding to 1.7% of chloride; and from 0.70 to

0.75% by weight of the pure chloride was injected. As many ties shattered and split under traffic, it was concluded that the strength and quantity of the injection was too great, and this has been gradually reduced until the strength is now represented by 2½°, and the quantity is from 0.50 to 0.55 per cent. In the author's opinion, this is probably a mistake. In Table No. 9 it will be noted that of 92 206 ties removed from six districts during 2 years and 4 months, 87% were rotten, and only 11% were removed because of shattering. Of course the limiting line between decay induced by shattering under the rail and that acting on the tie generally is difficult to define, and these figures may not fairly represent the relative difficulties from the two causes. This trouble with shattering has been found elsewhere with different material and with very much higher strengths and larger quantities of the chloride, without, however, developing into any very serious matter. The German practice at some works is to use a solution 3° strong, and the specifications for treatment at the J. Ruetgers Works require, in case of green or close-grained wood, into which it is impossible to inject the required amount of solution of 3° strength, that this strength shall be increased even up to 4½°, so as to secure the presence of an equivalent amount of the zinc chloride in all ties. The Chicago Works use at times a 3.8° solution and now inject 1% of chloride; and yet, out of 4 000 000 ties treated, not one broken tie has been reported. The proportion of those shattered to pieces is unknown, but is believed to be small.

The best German expert advice is to the effect that the change in strength from 2½ to 2½° is entirely too small to affect at all the strength of timber. If the shattering at any time exceeded or nearly approached the destruction by rot, it was more probably due to the use of excessive heat or impure chloride.

"With few exceptions the treated ties have been used exclusively in the main line between Houston and El Paso, a distance of about 836 miles." Tables Nos. 6, 7 and 8, give very complete data as to the life of ties on the Atlantic system, which includes the above.

To obtain the information regarding treated ties "it has been the practice of the company to stamp the ends of the ties, immediately after treatment, with a brass die giving the month and year of treatment. Track foremen have been required to put the stamped ends of all ties upon the same side of the track, that is to say, on the right hand going from New Orleans to El Paso. Foremen are provided with

an especially prepared blank on which they are required to report at the end of each month, the number of treated ties of the same date, removed by them; also the causes of removals of these ties, that is to say, whether rotten, shivered or broken, or from miscellaneous causes. These reports are then forwarded to the office of the Engineer of Maintenance of Way, where they are consolidated."

TABLE No. 6.—Classification of Ties in Track, Atlantic System, Southern Pacific Railroad, June 30th, 1897.

Kind.	Total number.	Percentage of whole.
Cypress	3 051 476	49.6
Untreated heart pine	1 019 435	16.6
Creosoted ties, principally bridge ties.	42 804	0.7
Burnettized sap pine	1 704 012	27.7
California red-wood,	323 371	5.3
Oak	9 214	0.1
Totals	6 150 312	100.0

TABLE No. 7.—Tie Renewals, Atlantic System, Southern Pacific Railroad.

Year.	Miles of track including sidings.	Number of ties renewed of all kinds.	Percentage of removals to total in use.	Average percentage per year for 5 years.
1887	1 931.56	498 581	9.8	.....
1888	1 946.86	589 969	11.5	.....
1889	2 018.66	508 150	9.5	.....
1890	2 025.32	410 153	7.5	.....
1891	2 049.07	497 617	9.2	9.5
1892	2 063.18	495 287	8.8	.....
1893	2 085.32	428 053	7.3	.....
1894	2 128.65	365 393	6.1	.....
1895	2 175.97	430 014	7.0	.....
1896	2 192.87	431 628	7.0	7.24
1897*	2 198.28	231 460	3.8	.....

* For 6 months only, from January 1st, 1897, to June 30th, 1897.

The figures for removals in 1893 include the removals of previous years, separate figures for which were lost.

"It will be observed from Table No. 8 that nearly all the 1887 ties have been removed, whilst an unusually large number of 1888 have also been taken out. Our records for these years show that a large percentage of the removals was due to the ties having been shivered and broken."

This is attributed to the fact that during the first few years the timber was overheated, the temperature in the cylinders being at

TABLE No. 8.—Treated Ties Laid and Renewed, by Years, Atlantic System, Southern Pacific Railroad.

Year treated.	Number of treated ties laid in same year.	Year removed.					Total number removed.	Number of years laid in track in July 1st, 1897.	Percentage remaining in track July 1st, 1897.
		1893.	1894.	1895.	1896.	1897*.			
1887.....	4 144	472	1 020	1 156	1 401	95	4 144	10	0
1888.....	118 098	3 199	3 602	10 665	16 883	9 419	43 768	9	63
1889.....	196 365	528	547	3 337	5 545	3 867	13 824	8	93
1890.....	212 677	69	67	940	3 536	4 424	9 036	7	96
1891.....	180 670	.....	8	185	936	1 035	2 164	6	98,8
1892.....	247 179	.....	.....	46	336	444	826	.....	.....
1893.....	179 229	.....	.....	269	177	120	566	.....	.....
1894.....	190 769	.....	.....	6	18	79	103	.....	.....
1895.....	190 802	.....	.....	.....	129	1	130	.....	.....
1896.....	165 783	.....	.....	.....	.....	1	1	.....	.....
1897	86 737	.....	.....	.....	.....	.....	.....	.....	.....
Total number removed.....		4 268	5 244	16 604	28 961	19 485	74,562	.....	.....

* These figures are for six months only, from January 1st to June 30th, 1897.

times as high as 300° Fahr.; and also to the strength of the solution. The latter has already been commented upon. The average life of the ties treated in 1887, considering the 472 reported as removed in 1893 as actually coming out that year, is 8 years, the amount of error from this assumption being small. Of the later years the average life of the ties treated cannot now be figured, as many of them are still in service. Of the 1888 ties 63% were still in track after 9 years' service; of the 1889 ties, 93% after 8 years' service; of the 1890 ties, 96% after 7 years' service.

It is not claimed that the decrease in the percentage of renewals of ties of all kinds during the past 10 years from an average per year of 9.5% for the years 1887–91, to an average of 7.24% for the years 1892–96, is due solely to the use of the preservative process; but, in the opinion of Mr. Kruttschnitt, that is an important factor in it. He says:

"Many years ago I became convinced that the greatest economies that could be effected in railroad operation would be those effected by the treatment of timber. From the year 1885 when I assumed charge of the Atlantic System, I have been consistently working to the policy of using nothing but Burnettized or creosoted ties, or redwood and

cypress. The California redwood, when protected against crushing, is practically everlasting, and our statistics show the gradual increase in the number of what we might call permanent ties, and a gradual decrease of untreated pine and oak. We hope within the next 3 or 4 years to make the latter disappear entirely. The reduction in the number of ties renewed per mile is due to three causes:

"*First.*—Treatment of timber.

"*Second.*—Reduction in the use of heart pine and increase in the use of redwood and cypress.

"*Third.*—To tie-plating."

Quoting again from Mr. Mahl's report:

"The objection to the Burnettized ties, as we have treated them, is that moisture will dissolve the chloride around the ends and outer surface of the ties, thus leaving an outer shell unpreserved, the timber in the same having resumed its natural condition, which, for the class of ties we use, is poor, indeed. The extent of this washing out of the zinc is dependent upon the annual rainfall, the quality and cross-section of the ballast used and the efficiency of the drainage."

The effects of moisture on the ties in the different sections being merely conjectural, beginning in 1896 the records were so arranged as to permit each roadmaster's district to be studied independently. The results so far obtained can be considered as indicative only, but they are interesting.

The Glidden District has a length of main line of 86 miles, beginning at Houston, Tex., and extending west to Glidden. It runs through a low flat country, and is ballasted with coarse gravel which extends 3 ins. above the ties at the center of the track and slopes so as to leave the ends of the ties free. The ballast is 8 ins. thick under the ties. The average yearly rainfall at Houston is 49 ins., which is a fair allowance for the entire division. A 61½-lb. rail with a 4¼-in. base is used.

The San Antonio District joins the preceding at Glidden and ends at San Antonio, Tex. It has a length of main line of 121 miles. The profile is undulating, the line crossing the drainage of the country. A 61½-lb. rail, with a 4¼-in. base, is used. The ballast is the same as for the Glidden District, and the roadbed is well drained. The average yearly rainfall at San Antonio is 29.5 ins., and 30 ins. can be assumed as the precipitation for the San Antonio District.

The Del Rio District has a length of main line of 170 miles. The profile is undulating, and there are some grades of 1 per cent. The roadbed is of standard cross-section, well drained, and is ballasted

with coarse gravel. A 50-lb. rail, with a 3⅞-in. base, is used. The rainfall for the District is given by the Government Report on Climate, for 1896, as between 20 and 30 ins.

The Sanderson District has 138 miles of main line, and runs almost entirely through mountainous country, with maximum grades and curves of 1% and 10°, respectively. The rail is half 60-lb. and half 50-lb. There is no ballast, the ties lying on the natural bed of soil and rock. The rainfall is less than for the Del Rio District, being given in the Government Report as from 10 to 20 ins.

The Valentine District has 154 miles of main line, through country of the same character as the Sanderson District. The rail is about half 70-lb. and half 50-lb. The track is ballasted for 42 miles with fine gravel, and is of standard section. The rainfall given is the same as for the Sanderson District, but is probably nearer the lower limit.

The El Paso District has 161 miles of main line through country of the same character as the two preceding sections. The rail is half 61½-lb., and half 50-lb. There is no ballast. The rainfall is light, and about 55 miles of the line is through arid country; probably 8 to 10 ins. precipitation would be an average for the District. The removal of treated ties only, on these various districts, for the 28 months ending April 30th, 1898, is given in Table No. 9.

TABLE No. 9.—Removals of Burnettized Ties and Causes thereof on Six Divisions, Atlantic System, Southern Pacific Railroad, from January, 1896, to May, 1898.

District.	No. of ties removed.	Cause of Removal.						Average life of ties removed.
		Rotten.		Shivered, broken.		Accidental and miscellaneous.		
		No. of.	Per cent.	No. of.	Per cent.	No. of.	Per cent.	
Glidden	6 048	5 542	92	506	8	0	0	6 yrs. 9 mos.
San Antonio	41 691	36 750	88	4 691	11	250	1	8 " 2 "
Del Rio	15 564	14 247	92	1 240	8	77	.....	8 " 10 "
Sanderson	2 607	1 716	66	145	5	746	29	6 " 8 "
Valentine	12 899	10 911	85	1 825	14	163	1	7 " 0 "
El Paso	13 397	10 898	81	1 596	12	903	7	7 " 6 "
	92 206	80 064	87	10 003	11	2 139	2	7 yrs. 11 mos.

It should be noted that the average life given in the table is that of the ties removed, and is of little value as indicating the probable average life of a lot of ties unless the ties have been in the track long enough to have covered the life of the most durable; and instead furnishes only a minimum life. To illustrate this, Table No. 10, covering additional data for the first two districts, is given. It will be noted that the average life of the ties removed is increasing steadily. The discrepancies in the figures for the average life in the two tables are due to the first covering 28 months, while in the second the averages are for each year.

TABLE No. 10.—Removals of Burnettized Ties from Two Districts, Atlantic System, Southern Pacific Railroad.

Year.	Ties Removed from Track.		Average Life of Ties Removed.	
	Glidden District.	San Antonio District.	Glidden District.	San Antonio District.
1894....	848	4 109	5 yrs. 8 mos.	6 yrs. 2 mos.
1895....	841	10 330	6 " 5 "	6 " 10 "
1896....	1 871	16 904	5 " 11 "	7 " 9 "
1897*...	1 750	8 186	7 " 5 "	8 " 2 "
1898....	3 637	19 986	7 " 0 "	8 " 8 "

* 6 months.

In Table No. 9 the average life on the Sanderson District is much lowered on account of the large number of removals due to accidents and miscellaneous causes. On the Valentine and El Paso Districts the average is also lowered by extensive rail renewals during the last 3 years. Mr. Mahl's opinion is "that the relative difference in life shown to exist between the Glidden, San Antonio and Del Rio districts may be reasonably taken as representing the relative life of ties for those districts." Mr. Mahl states: "With few exceptions the treated ties have been used exclusively in the main line between Houston and El Paso," being composed of the Divisions previously described. By comparing Tables Nos. 8 and 9, it will be seen that the average life of all the ties removed on all the divisions in the 18 months ending July 1st, 1898, was 7 years and 11 months; and that on July 1st, 1897, of all the ties laid 8 years previously, 93% was still in the track.

"The track work on these districts during the period in question consisted by far of only ordinary section labor engaged in the repair

and maintenance of the roadbed and track. For some years past, our repairs of roadbed and track over the entire system have shown gratifying decreases each year, until, for the present year ending June 30th, 1898, these repairs will not exceed $305 per mile of main line for the part west of Houston. This is the lowest cost for repairs of roadbed and track that has ever been obtained, and it is in part attributable to the increased percentage of treated ties in track."

The results of Burnettizing were so satisfactory on the Atlantic System that a plant was built in 1894 for the Pacific System of the Southern Pacific Company, and, as the timber supplies were widely separated, for economy the entire works were placed on wheels, and when a change in location is necessary, the plant forms a train which has successfully traversed 16° curves and 3% grades. It consists of two retorts, 6 ft. in diameter and 114 ft. long, and has a capacity of 2 500 to 3 000 ties in 24 hours. The total number of treated ties in the tracks of the Pacific System, up to the end of 1897, was 1 448 315, all but 74 012 of which were placed subsequent to 1894; consequently no information is yet available as to their life. This particular plant is described in a paper* by W. G. Curtis, M. Am. Soc. C. E., and was built under patents owned by Messrs. Curtis and Isaacs, both of the Southern Pacific Railroad.

A similar plant is now under construction for Mr. O. Chanute, to treat ties for the Chicago and Eastern Illinois Railroad.

Summary and Discussion of Recent American Experience.

From the preceding, the experience of American roads with treated ties may be concluded to be generally favorable. The Atchison, Topeka and Santa Fé Railroad officials, after 12 years' trial on a large scale, believe they are getting from 11 to 12 years' service from mountain pine, having a natural life of about 4 years; while from natural white oak they get but 6 years in heavy main line service; and from cedar 10 years under light service.

The Union Pacific Railroad reports about 9 years for mountain pine and spruce. The Chicago, Rock Island and Pacific Railroad report of the count of ties now in place, indicates a life of at least 8¾ years. The unimpeached counts of removals prior to 1894 show that in that year 84.7% of the ties laid 7 years previously was still in track. The record of the four sections just west of the Missouri River, as here fig-

* *Journal* of the Association of Engineering Societies, July, 1895.

ured, indicates a life of 11 years. The experience of the Pittsburg, Fort Wayne and Chicago Railroad with hemlock and tamarack ties, only 1% having been removed after 7 years; and of the Duluth and Iron Range Railroad, with white and Norway pine and tamarack which, after 7½ years' service and after having been taken up and relaid once, were still in nearly perfect condition; furnish conclusive evidence that the treatment is efficacious, and presumptive evidence that a life of 11 or 12 years may be expected. The discovery of about the same number of treated ties in the tracks of the Rock Island and Peoria Railroad, as were laid in 1891—at which time the last were laid—demonstrates how entirely mistaken were the officers of that road as to the life to be secured from such ties, even under the unfavorable conditions existing as to drainage and ballast.

The Southern Pacific Railroad, operating on the most perishable material, with a life of only 3 years at the best, had 93% of the ties treated and laid in 1889 still in the track after 8 years. The record for the ties treated during the 2 previous years was not as good, for known reasons; however, those laid during 1887 gave 10 years' service, and of the 1888 ties, 63% was still in use 9 years later.

The woods which have been treated and used for a considerable time in regular service are the mountain pines, firs and spruces, hemlock and the sapwood of the yellow pine. The woods used experimentally are the cottonwood, sweet gum, red and black oaks, beech, tamarack and Norway and white pine. Of other timbers which have not been tested, it can be expected that beneficial results will be obtained from treating any wood which will take the treatment well; that is, into which the proper amount of chloride can be injected. This is determined by the physical characteristics of the timber, the size of the sap ducts, the thickness of their walls and the amount of gummy substance contained in the sap. It does not follow, however, that equally good service will be secured from two materials, even if they take treatment equally well. The table on page 304 shows that in the German practice their beech and pine are treated with the same quantity of chloride, but the pine has 3 years' longer life, if both are Burnettized. Treated with tar oil, the beech is apparently the longer lived by 50 per cent.

The German practice is to treat only thoroughly seasoned wood. This is generally considered to be desirable in America also, but seems

to be unattainable. Mr. W. G. Curtis, however, in his paper describing the practice of the Southern Pacific Railroad, Pacific System, already referred to, states that freshly cut ties take the treatment much more readily than those partly seasoned; so that if the ties are freshly cut, they can get six charges through two cylinders in 24 hours; while if partly seasoned, they get but five charges.

"We find the time required varies greatly with the kind of timber and with the time during which the ties have been seasoning. California mountain pine, fir and spruce require less time than Oregon fir, and all timbers are more readily treated when freshly cut. An Oregon fir tie seasoned in the air for 2 years will take double the time for the treatment required for one freshly cut. Occasionally a close-grained, well-seasoned tie will not receive the preservative at all, the fluid penetrating into the sides only about $\frac{1}{2}$ in."

While this statement is probably correct for the particular timber treated, it is entirely contrary to the practice at other places, both here and abroad. As already quoted at the Houston works of the Southern Pacific Railroad, "if green ties and ties which have been seasoned for 3 to 6 months were subjected to the same treatment, the percentage of absorption of the seasoned ties would be much greater than the absorption of the green ties." The Chicago Tie Preserving Company finds it impossible to get satisfactory work with freshly cut ties, whether of hemlock, beech or tamarack. During 1898 this company ceased operating entirely for nearly 2 months, and piled up 70 000 ties in the yard at considerable expense for rehandling, to permit them to season sufficiently to take the preservative. These peculiarities of timber, however, are not necessarily dangerous. It is simply necessary to fit the treatment to the material; and emphasizes two things: That such a cost should be allowed for treatment as to permit of occasional exceptional expense without loss, and that persons entrusted with such work should be above sacrificing the quality of their work for a small immediate gain.

The Chicago Tie Preserving Company has very kindly given access to all its notes and records, both of routine and experimental work; and by request has submitted the following statement:

"This company has made hundreds of experiments upon thousands of ties. These were tried partly by weighing individual ties before and after treatment in the regular runs, to ascertain the absorption; and partly, since 1894, by special treatments in an experimental retort, holding three ties, wherein various methods could be tested without

interfering with the regular runs. The objects in the latter were two-fold:

"*First.*—To ascertain by what modification the largest quantity of chloride of zinc could be injected without making the timber brittle. This has been attained, the amount of antiseptic absorbed since 1896 having averaged double that injected in 1886, with what results remains to be seen, while not a tie has been reported as having broken in the track.

"*Second.*—To ascertain whether the ties were in proper condition to treat, and the idiosyncrasies of various woods.

"As regards the conditions, it has been found that ties delivered early in the spring require to be treated differently than those arriving in the late summer. Sometimes the former are quite unfit and must be placed upon the ground to season. This should preferably be done in cribbed piles (when there is space enough) to allow the air to circulate between the ties. The same has to be sometimes done in the autumn, when the ties have been rafted or floated out to the vessels.

"Refractory lots of hemlocks are piled upon the ground, and a few tested month by month to ascertain their condition, It is found that they gradually improve in receptivity, so that in 6 months they absorb two to three times as much as upon their first arrival. This agrees with European experience, where ties are seldom treated less than a year after being cut, but it is the reverse of the experience of Mr. W. G. Curtis, on the Southern Pacific System in California and Oregon," as quoted above. "The same claim was made by Mr. O. Thilmany, who stated: 'We prefer green timber; dry timber has to be steamed longer. Only live and sound timber can be preserved.'

"Perhaps these puzzling contradictions are due to differences in the character of the woods treated, or in the time of cutting.

"It is well recognized that natural woods are more lasting if cut when the sap is down. Summer-cut hemlock ties tested repeatedly by the Chicago Tie Preserving Company only absorbed about half as much as those winter-cut, and it was also found that some gum and pine ties cut in Virginia, North Carolina and Georgia in September, absorbed from two to eight times as much as ties from the same woods cut in April.

"It is most desirable, before treating a tie, to know when it was cut, and an ideal condition would be to have the ties identified as to the time of their cutting, and treated in the same order.

"After treatment all ties should be allowed to dry, as is always done in Europe. Several lots were treated twice in Chicago, having absorbed in the first treatment but about 14% of their weight. All of this gain they lost in 5 months of seasoning, and 12% more; and were then in a proper condition for a second treatment.

"As regards the idiosyncrasies of various woods, it has been ascertained that hemlock and beech are much more refractory than pine, principally early in the season; that white and burr oak should not be Burnettized, but that black oak, red oak and water oak take the treatment very well, and will outlast Colorado pine. Cedar is hardened by the process, but not enough to pay for the cost of injection. Tamarack, like spruce, is an uncertain and variable wood. Some ties cut at the same time, from the same forest, were found to vary as 2 is to 1 in average absorption. This was attributed to the varying amount of resin in various trees, and after many experiments it was learned to distinguish this difference by the eye, but, as it was impracticable to sort the trees before cutting them down, the injection of tamarack was finally abandoned altogether.

"There were also great differences in the absorption of individual ties of the same age and same species of wood. In extreme cases some will take eight times as much as their neighbors, and it becomes a serious problem how to regulate the injection. It will not do to treat all kinds of wood alike, and, when a new species comes to the works, many experiments have to be tried before it is known how it is best to deal with it.

"The operations of the works are, of course, controlled by an elaborate system of measurements and accounts, so that it is known accurately how much of the antiseptics are absorbed by each run of, say, 400 ties, but it is not known how much each individual tie has taken, save in special experiments. Changes are in contemplation to circumscribe this uncertainty, and in this, as well as in other details, we shall have to profit by the experience of the Europeans, who have been preserving ties for about 50 years, and have materially improved upon their original methods."

In examining the records of tie treatments it is necessary to recognize the fact that the life of the exceptional tie, whether long or short, is of no value, except as a curiosity, and as containing a possible lesson in detail. As an instance, a tie was exhibited in the Paris Exposition of 1889 which had lasted 46 years in the track of one of the French railways. It had been prepared by the sulphate of copper (Boucherie) process, but nevertheless this process has been generally abandoned in favor of others. So far as the value of a treatment itself is concerned, such records of individual ties may be worse than useless, leading to an expectation of length of life which cannot be attained by the average, or discouraging the investigator without reason. The valuable thing is the average durability, and by this only can any results of real value be obtained.

The author has been impressed, while reading the statements of 10 years ago, with the fact that the advocates of preservative methods claimed too much. The life of treated ties was unhesitatingly put at 15 years, a limit which the occasional tie has reached, and which indeed, under favorable circumstances, the foregoing records show to have been reached by a considerable number of ties, and to which, with future treatment, as the operators become more skilful in the selection of material and its application, perhaps the average life may approach. The record of commercial use in the United States has failed to reach any such period, and it is entirely unnecessary that it should, to make the treatment desirable and economical.

The author believes his investigation has demonstrated that the cheaper woods, hitherto unused, are made fully as durable or more durable than white and burr oak, by Burnettizing; and this fact is sufficient to make the adoption of the treatment desirable for many roads, if indeed it may not be said for American roads generally.

One point this inquiry has not solved is the relative advantages of the zinc-tannin or Wellhouse process, and the old method of Burnettizing, using nothing but the zinc chloride. This hinges upon two points: The solubility of the chloride when deposited in the wood, and the efficiency of the chemical action between the glue and the tannin, required to form the particles of leather which are supposed to plug up the ducts. As to the first, there is general evidence that part of the chloride can be washed out of the timber, although it is not easily done. Mr. J. D. Isaacs, of the Maintenance of Way Department of the Southern Pacific Company, in a paper* states:

"The author thinks this objection is greatly exaggerated. He has attempted to leach this chloride out of timber with running water, but could extract only an insignificant proportion, even when the timber was in fine shavings. All the chloride could not be taken out of chips from treated timber by boiling in repeated changes of water. If leaching takes place, it is certainly very slowly."

The above applies to wood treated with the chloride only. Unfortunately, this railroad was unable to furnish any chemical analyses of treated timber which had been in service for a number of years. That the chloride is gradually leached out of material treated by the zinc-tannin process is probably true. A number of analyses of timber after years of exposure shows considerably smaller quantities of zinc

* Thirteenth Report of the California State Mineralogist, 1896.

than it is believed they originally contained. One such instance has been given in connection with the Union Pacific ties, where only about one-fourth as much of the chloride was found as was generally injected. The evidence at hand would be absolutely conclusive if it were possible to know how much the ties which were analyzed contained originally; but this is not the case, as the amount of absorption of a retort full of ties is as near an individual record as it is now possible to go; and it is possible that the tie analyzed might have been one which was refractory and took very little when treated. If such had been the case, however, the tie would probably have been of short life; and it is safe to conclude that after a certain number of years, depending upon the rainfall and the efficiency of the track drainage, the amount of chloride remaining in a tie will be so reduced as to be of no further value, whether the treatment be with zinc alone or in combination with the glue and tannin.

If the claims for the formation of the artificial leather by the tannin treatment are correct, and it is understood that this is verified by microscopic examination, it is probable that the leaching is delayed. That the treatment does not prevent the passage of moisture into the wood is evidenced by the rapidity with which the treated timber seasons, as well as by the loss of the zinc shown by the analyses. The results in future on the Atchison, Topeka and Santa Fé Railroad may determine the relative advantages of these two systems, as there are large numbers of ties now in use there treated by each. To insure the greatest success by either, however, it is believed that the injection of large amounts of chloride must be practiced, and, generally, this can be done only by using timber at least partly seasoned; and the subsequent partial seasoning before being placed in track.

Granting that certain woods may take the antiseptic in sufficient quantities even if treated green, so that it may not be accurate to say that it is necessary to good work that at least partly seasoned wood should be treated, there are certain other conditions which can be said to be essential to good work. These are pure zinc, pure acid, the absence of any free acid in the chloride solution, the use of a heat not exceeding 250° in the retorts, and sound, live timber to operate on. To treat dead or dozy wood is to waste time and chemicals. Chloride of zinc apparently has no power to stop decay which has already begun, and it is doubtful if any treatment is better in that

respect. To these may be added faithfulness and integrity of a high order on the part of those operating the works.

The output and profits of a plant can be very largely increased by the lack of the last, and the results of such lack will not be apparent until years afterward.

To make the later identification of the ties certain, so that the life realized can be determined, it is believed that nothing will prove as satisfactory as the use of dating nails, which are nails with large heads in which is stamped the current year, and which are driven in the center of the top surface of the tie when laid. The cost of such nails is but small, and they are more accessible and certain than any substitute.

Foreign Practice.

The author regrets that he cannot furnish very much on the foreign practice of recent date. In France and Great Britain, the creosote process is used almost exclusively. The Burnettized timber was apparently not satisfactory. Creosote is comparatively cheap, and the increased life of ties so treated is considered as worth the additional cost over Burnettized ones.

The roads in the German Railroad Union, however, use very generally, either zinc chloride or a combination of the chloride and creosote, as well as creosote alone. Impregnation with the chloride alone has decreased while that with the mixture of chloride and creosote has largely increased. Beginning in 1895 all the pine ties used on the Prussian State railroads were treated by the zinc creosote process. This consists of the injection of the same amount of zinc chloride, with which is mixed creosote, in the amount of $1\frac{1}{4}$ lbs. to the cubic foot of timber; the mixing being done by a peculiar process. It is claimed that the results of this treatment are much superior to those with zinc chloride alone. In 1885, the Ruetgers Works furnished one road with 171 000 pine ties treated with zinc creosote, on a 9-year guarantee. In 1894, at the end of the 9 years, only twenty-nine had become unfit for use and none of these was rotten. Mr. Chanute is about to go to Germany to investigate this process, and it is hoped that he may be able to contribute the necessary data to the discussion of this paper, so that the value of the treatment may be determined. The additional cost over the regular Burnettizing process varies with the price of creosote, but is about 4 cents per tie. The zinc-tannin process has never been used outside of the United States.

The Ruetgers Works treat ties by Burnettizing on the following specifications: The solution of zinc chloride is to be 3° Beaumé strength at a temperature of 14° Réaumur. Ties and timber are to be treated until the absorption of seasoned timber is for

Pine wood.....19 lbs. per cubic foot.
Oak " .... $6\frac{1}{3}$ " "
Beech " ....19 " "

" If the amount of liquid required cannot be injected in the manner prescribed because the wood is too green or is exceptionally dense, the liquid is to be strengthened in such a manner that the proper amount of water-free chloride of zinc is taken up as prescribed, namely, that to be contained in a 3° Beaumé liquid.

" From this it follows that when, for instance, only 200 kilograms can be introduced into 1 cu. m. of pine wood, the chloride of zinc bath must be made $4\frac{1}{2}$° Beaumé."

The German report on railroads previously referred to gives certain tables from which the following are obtained:

TABLE No. 11.—Costs and Results of Wood Preserving for the Union of German Railroads for 1896.

Kind of tie.	Cost of crude tie.	Treated with Chloride of Zinc.					Treated with Tar Oil—Creosoted.				
		Absorption in pounds.	Cost of treatment.	Total cost.	Average duration, in years.	Cost per year.	Absorption, in pounds.	Cost of treatment.	Total cost.	Average duration, in years.	Cost per year.
			Cts.			Cts.		Cts.			Cts.
Oak....	$1.49	24.2	13	$1.62	15	10.8	{15.4 24.3	21 29	$1.70 1.78	24 28	7.1 6.3
Beech..	1.01	34	15	1.16	9	13.0	{66 79.2	50 59	1.51 1.60	30 34	5.0 4.7
Pine....	0.84	34	16	1.00	12	8.3	{50.6 79.2	43 57	1.27 1.41	20 23	6.3 6.1

This is based on a tie $6\frac{1}{2}$ ins. × 10 ins. × 8 ft. 10 ins.

From Table No. 12 it will be seen that in 1896 about 60% of all the wood ties in the tracks was impregnated in some way; and that all of the increase in number of ties in track over 1894 was in the treated ties. Unfortunately, it is impossible to determine the number treated by each process in use by these roads, and the figures are of value only as indicating the general use of treated timber in preference to the crude.

As a matter of interest, a line is added showing the amount of track laid with metal ties or sleepers.

TABLE No. 12.—Statistical Report of the Railroads of the Union of German Railroad Administrations for the Fiscal Years 1896, 1895, 1894.

Designation of railroads.	Year.	Abstract of Wood Ties, Laid in Track. Number of wood ties. Oak.	Beech.	Larch.	Pine.	Total.	Of the ties there are: Impregnated.	Crude.	Length of track laid in metal permanent way. Kilometers.
German, State and private roads	1896	28 279 084	3 248 882	92 585	36 947 827	68 568 378	59 834 087	8 734 291	22 805
	1895	28 635 001	3 283 867	91 924	34 775 635	66 786 427	57 626 304	9 160 123	23 102
	1894	29 353 375	3 132 261	102 936	32 241 230	64 829 802	54 041 505	10 788 297	20 868
Luxemburg, Netherlands and other Union roads.	1896	8 769 969	49 551	.....	541 285	12 009 368	1 718 815	6 730 887	495
	1895	8 281 293	48 392	.....	538 330	11 566 538	1 597 308	6 360 733	497
	1894	7 916 896	39 083	.....	552 105	11 200 509	1 552 809	6 046 430	495
Austria, Hungary and Bosnia roads	1896	31 533 486	5 406 009	4 066 135	6 144 072	47 149 702	18 197 013	28 952 689	404
	1895	30 192 026	5 437 128	3 913 322	6 056 466	45 598 942	17 755 877	27 843 065	368
	1894	29 286 099	5 797 985	3 581 179	5 778 177	44 443 440	16 882 687	27 560 753	366
Totals of all roads	1896	68 582 539	8 704 442	4 158 720	43 633 184	127 817 448	79 749 915	44 417 867	23 704
	1895	67 108 320	8 769 387	4 005 246	41 370 431	123 951 907	76 979 489	43 363 921	22 967
	1894	66 556 370	8 969 329	3 684 115	38 571 512	120 473 751	72 477 001	44 395 480	21 858

Cost of Treatment.

It is not proposed to enter into any discussion of the economy of, or the savings to be effected by, the use of treated ties. This is something which will vary with almost every road, being governed by the cost of white oak ties, cost of the inferior tie to be treated and location. It has been attempted to determine what can be expected from the treatment and with the cost thereof given below, it can readily be determined which is the cheaper—a white oak or a treated tie, and, in the author's judgment, that is what the question resolves itself into.

Apparently the evidence is conclusive that a treated tie is superior to a white oak; but, if only equal to it, on the basis of cost in most locations the treated tie will prove the more desirable.

The cost of Burnettizing sawed pine ties 6 ins. x 8 ins. x 8 ft. on the Southern Pacific Railroad in 1893, was about 10 cents each; while in 1897

it was a little over 6 cents. This includes nothing for interest on the investment, or depreciation; however it is for a very small tie, containing only $2\frac{2}{3}$ cu. ft.; and for an amount of zinc chloride only about one-half that required by the German practice. On the basis of the cost in 1897 (which amounts to $2\frac{1}{4}$ cents per cubic foot), the use of the standard German quantity to be injected and a hewn tie of $3\frac{1}{2}$ cu. ft., this cost would be about 12 cents per tie, or $4\frac{1}{2}$ cents per cubic foot, including nothing for interest or depreciation.

On the Atchison, Topeka and Santa Fé Railroad the cost of the zinc-tannin treatment, when the works were first built, was about 15 cents; in 1892 it was about 14 cents and 13 cents for Burnettizing (zinc chloride only). In 1897 the cost (zinc-tannin) was reported as 11.6 cents; no interest or depreciation being included. It is understood that the charge made by the Chicago Tie Preserving Company is from 16 to 20 cents for the zinc-tannin treatment, covering, of course, interest and depreciation as well as profit.

The contract prices in Germany for Burnettizing are: For pine and beech, 5 cents per cubic foot, and for oak, 4 cents; for treating with zinc-creosote, 6 cents for beech and pine, and 5 cents for oak; for creosoting, 15 cents for beech and pine, and 9 cents for oak. In creosoting, the amount of creosote per cubic foot is 12 lbs. for pine, 15 lbs. for beech, and $4\frac{1}{2}$ lbs. for oak. It is understood that the oak referred to corresponds to American white oak and not to the American red and black oaks, which will absorb as much as either pine or beech.

DISCUSSION.

Mr. Lowinson.

Oscar Lowinson, Jun. Am. Soc. C. E.—There is no question of greater importance to the engineer using wood, under conditions subject to decay, than that of the preservation of timber. A technical knowledge of the composition of the timber, of the chemical and mechanical conditions to be met, of the effects of moisture and frost, of wear and tear, of the juxtaposition of other materials, such as iron, steel or zinc against wood, of the resistance of the material to transverse crushing, rust or rot, and the relative cost of the operation, etc., is requisite for a satisfactory answer as to whether any method, or which method, is advisable for the preservation of timber.

Owing to the great expense and trouble of making continual physical and chemical analyses, the almost impossibility of examination after application, the long period of time required to determine the success of a method, and the necessity of having perfect confidence in the integrity of those who supply the materials and apply them, the use of some method of preservation of timber is, and for years will be, only on works where methods of investigation are carried out with care, or where the user places perfect confidence in the integrity and knowledge of the parties applying the process. Failure is due to ignorance and corrupt practice as much as to fundamental error of theory.

As the author states, there has been practically no English and little German literature on the subject of value, since the report of the Committee of this Society in 1885. Such knowledge as has been obtained is in the note books of the chemists and engineers who have investigated the question. The various methods of application of preservative processes have had an almost uniform experience. "The history of engineering is strewn with wrecks of numerous methods which promised much," but, under conditions of experience, performed little. Very nearly every large railroad in the United States has at one time or another experimented in this line, and many have ultimately given up, either because of the poor results of the experiment, or for reasons beyond the control of their engineers or chemists. There has been practically no advance in this department of applied science.

"The substances which thus far have given the best results are the application of chloride of zinc, the bichloride of mercury and the products resulting from the destructive distillation of coal tar" (used in creosoting).

The literature of Preservation ably discusses these processes as applied previous to the time of the above-mentioned report. A careful reading of the author's paper will show practically all the defects

Mr. Lowinson. and objections to the first and second processes. It will only be necessary here to add the practical difficulties which have hitherto prevented the more general application of these processes.

Practical Considerations Affecting the Adoption of Preservative Processes.

1. The Consumer.—In the everyday life of railroad maintenance the "powers that be" are appointed in most cases because of their ability to give their road a good showing on the stock market. The necessity of showing at all times a maximum excess of receipts over expenditures generally influences these representatives of the stockholders to object to the added expense of a plant, or the added cost of preserving the timber, which must thereby be entailed for the period of years until the entire track has been placed in good condition; even on the assumption that the process will be a success. It is assuming that an official would not be true to the trust held if he should increase the expense of maintenance for a single year and thereby depreciate the earning power of the invested capital, even though he might be certain of ultimate saving in annual outlay; because of the very condition, *viz.*, the property is a marketable one, and the only practical method of conducting the work is to make only such repairs as are absolutely necessary to prevent its ruin. The results of this process are seen in many large railroads. The only time of actual outlay is at the time of reorganization, or when the road passes into the hands of a receiver to whom the ultimate preservation of the property is of more importance than the question of temporary value of stock. In other words, "it is the duty of the official to run the road on a poverty basis even though at much greater ultimate expense."

2. The Process.—Besides those described, the creosote processes are the only others with which satisfactory results have been obtained. Referring to the report of the Committee of this Society, the statement, that creosoting is the most perfect method of preserving timber, still holds true. Since the publication of that report, and as the result of the investigations by the German Governments on their railroads, the attention of scientists has been devoted to the production of creosote and other products in such combinations as to be applicable without the expense of a plant for impregnation under pressure. The present commercial standing of the creosote process itself is also unsatisfactory because of its unsuccessful application in many cases, due either to insufficient impregnation or to the use of an inferior material. It may not be improper in this discussion to state the kind of creosote to be used and its composition.

In the first place, the creosote to be used is what is commercially known as coal-tar creosote. It is one of the many products derived

from the destructive distillation of coal-tar. From this are derived products covering almost the entire range of human application. Among these may be mentioned all the aniline colors, the various perfumes, essences and flavoring extracts, high-power explosives, medicines, fertilizers, saccharines, photograph developers, benzines, ammonias, antiseptics, lamp black, lubricating oils, varnish, creosote, shellac, cresylic acid and other wood-preserving chemicals, etc. Besides this material, there is a wood creosote, derived from the destructive distillation of pine, of which more will be said later. Mr. Lowinson.

The creosote must be a heavy dead oil, of the best quality, free from tar and other impurities. The ease of adulteration of this product by a mixture with naphtha, and the difficulty of detecting such adulteration without continued analyses of the material, are the probable causes of failure in most attempts where it was assumed that the genuine product was used. So extensively is the adulteration carried on that few engineers have confidence in the process because of their inability to secure this product. It is the cost of this creosote that prevents its practical application. It is necessary to know the constituents of the coal-tar product, and, at the present time, commercial reasons require that it be brought from Europe. Its composition depends upon the conditions of its manufacture. It must be dense, and have a specific gravity varying from 1.05 to 1.20. It must distil at a temperature of not less than 425°, and must be rich in its principal ingredients, naphthaline (C_{10} H_x) and anthracene (C_{14} H_{10}).

The method of impregnation, when done by outside contractors, is such as frequently prevents the inspector from knowing whether the proper material is really being injected. It is not the speaker's intention to question the integrity of any plant, but the experience of many engineers has been that deception has been practiced here to such an extent as either to drive out of competition many reputable concerns, or to so discourage the user as to cause him to despair of success. Not only is this true, but the chemical analysis of the material under inspection is a difficult process. These objections are applicable to all processes of impregnation. Properly executed, however, the results are gratifying to the engineer.

The New York Division of the Erie Railroad, 10 years ago, had ties Burnettized, and whereas, untreated, their average life was 8 years, most of these treated ties had to be replaced within 6 years. The zinc-tannin process, with the addition of glue, appeals to the thoughtful engineer as meeting most of the conditions required for a preservative.

3. The Material.—The speaker would ask the author what the depth of penetration is and what method, if any, has been adopted for the protection of the tie when cut or bruised so as to admit water and frost; also the effect of checking and the effect on seasoning of timber.

The speaker does not agree with the author that the transverse

Mr. Lowinson. crushing is a negligible quality. A series of tests made by the speaker* on the transverse crushing strength of timber gave the following ultimate values: For seasoned white pine, 550; yellow pine, 774; and oak, 1 200 lbs. per square inch.

In an interesting report on the "Strength of Bridge and Trestle Timbers" by a committee of the Association of Railway Superintendents of Bridges and Buildings, of which Walter G. Berg, M. Am. Soc. C. E., was chairman, October, 1895, in regard to transverse strength, the Committee reported and recommended as the average safe compressive strength across the grain:

Southern yellow pine.....	350 lbs.	per square inch.
Chestnut and Norway pine.	250	" "
Spruce, cypress and cedar.....	200	" "
Oak	500	" "

The stresses to which a tie is subject, and which in turn contribute to its destruction, either through mechanical force or decay, are the transverse crushing resistance, torsion due to track moving and deflection of rails, the shock due to hammering by the live load as transmitted through the rail, and the elements. The results of the speaker's investigations have led him to recommend the adoption of a tie-plate between the rail and the tie whenever a preservative process is to be applied in any tie other than oak, for the very reason that, although the treatment for preservation will aid in lengthening the life of a tie, the decay is only one of the causes of destruction; and protection must be sought in every possible way. When it is seen that the safe transverse crushing strength, for instance, is only 250 lbs. per square inch, even allowing for alleviation due to stiffness of the rail, the speaker would urge as an additional improvement that the tie-plate be used.

In regard to cutting treated timber, although not so often met with in railroad ties as on street railroad or dock work, the speaker fears that it is not practicable to treat the timber in the way stated unless there be some arrangement for treating the exposed surfaces.

In the case of creosoted timber, the speaker has seen both the hot and cold dead oil of creosote applied with a brush. A street railroad in New York City is using a 6 × 6-in. "creosoted" yellow pine stringer under its heavy rails, supported on the yokes about 5 ft. apart on centers, and in a large number of cases it is found necessary to notch the stringers at the yokes. The speaker is not aware that the notched portions are undergoing any treatment, but whether or not because of this the creosoting to which they have been subjected will increase the life of the timber, time alone can tell.

Liquid Preservatives.—The necessity which has arisen in European countries for treating timber has brought about a considerable amount

* *The Engineering Record*, December 12th, 1896.

of experimenting in order to find a substitute for the expensive method of creosoting. There are two reasons for the great cost of treating by creosote. These are: Mr. Lowinson.

First.—The high melting point of the naphthaline (175° Fahr.) and the anthracene (210° Fahr.), which are the principal preservative chemicals of creosote.

Second.—The necessity of a pressure process to complete the impregnation.

There are now on the market in Europe at least two materials by the use of which it is claimed that these objections are done away with. Whether the statements of the manufacturers are true or not, the speaker is not yet prepared to state. He certainly would recommend the most exhaustive examination before adopting them. In one case it is claimed that by the addition of zinc chloride to the creosote (a combination of Burnettizing and creosoting), before the completion of the process of distillation, the chloride combines with the naphthaline (C_{10} H_8), to form yellow heavy oils, dichloride of naphthaline (C_{10} H_6 Cl_2) and chloride naphthaline (C_{10} H_7 Cl); and the anthracene (C_{14} H_{10}) is converted into dichloride-anthracene (C_{14} H_8 Cl_2) and chloride of anthracene (C_{14} H_9 Cl), which at temperatures of 100° to 200° Fahr. possess great viscosity and consequent penetrating power. The speaker has seen some remarkable experiments as to the penetrating power of this material.

Wood Creosote.—Wood creosote is a material obtained from the destructive distillation of pine. Its vital principle is paraffine as distinguished from the naphthaline and anthracene of the coal-tar creosote. A large number of so-called preservatives consist of this material, either by itself or in combination with other materials. It is not looked upon favorably as a preservative, because of its unstable composition and its solubility. The much-advertised creosote and other paints and shingle stains have this material for a basis. It is the speaker's opinion, however, that the addition of a paraffine to a paint adds nothing to its preserving power.

H. S. Haines, M. Am. Soc. C. E.—By far the largest part of the cost of operation of a railroad is devoted to wages and salaries, but if the part devoted to supplies be examined it will be found that the principal expense is absorbed in two items, fuel and timber. When the amount of ingenuity and time which has been devoted to devising means and methods for diminishing the cost of fuel per mile run is considered, and how little ingenuity has been devoted to endeavoring to extend the life of timber, it is evident that the two items are not held in proper proportion. There is hardly any subject which can come before this Society which has more economic value than the preservation of timber; and therefore the speaker has examined this paper with interest to see what could be learned with reference to the general Mr. Haines.

Mr. Haines. question of the preservation of timber, but confesses to a good deal of disappointment in finding that substantially no advance in knowledge on this subject has been made since the Committee of this Society made a report on the subject in 1885. The conclusion of that Committee was substantially that "creosoting was the most perfect process to use, but in view of the expense it may be preferable to use a cheaper process. With our present knowledge we believe the zinc-chloride process is the advisable one to use for ties in this country."

By reading this paper through it will be found that in the year 1899 we are no farther ahead, and know no more than in 1885. In fact, it is a still further disappointment to see that this paper, which has been prepared apparently with a great deal of care, the author having endeavored to gather all the information he could find on the subject, presents so little. It is all impressions, traditions and legends, with scarcely any substantial facts, except in reference to what Mr. Kruttschnitt has done. The speaker knows that Mr. Kruttschnitt is very reliable in all his statements, but unfortunately his statements only cover the period from 1894 to the present time.

In a mile of track there are about 2 600 cross-ties, costing, in the track, nearly 50 cents apiece, and their average life is about five years, which represents about $230 per mile per annum. If the life of these ties could be extended, doubled, for instance, the value of any process which would accomplish this can readily be seen.

Reference is made to what has been done in Europe, where certainly a great deal more attention has been paid to the preservation of timber than in America. The speaker's experience has been only with creosote, but extends over many years. Probably no one in this Society has had more extended experience in the use of creosote for timber on railroads than the speaker, and his conclusions are the same as those of the Committee, that it is to-day the most perfect process which can be used. The best evidence of the truth of this statement is that abroad, where the zinc-chloride and also the creosote treatment have been used for many years, they have come back partially to creosote; that is, they are using zinc chloride and creosote in combination, apparently with the view of getting a cheap substitute for the latter.

As to the use of creosote itself, of course every one has had disappointments. The speaker has had some very bitter ones. Mr. Lowinson is right in saying that if the creosote contains certain constituents, particularly naphthaline, and if it is used in certain ways, the result will be just as certain as anything known in our present experience of treating timber. There is not any doubt of it. But the so-called wood creosote is a fraud, nothing else, and any substance that is soluble in water is useless for the preservation of timber, just as anything that is proposed to be superficial in its application is of but little value, for the reason that wherever a notch is cut

in a piece of timber a surface is exposed to decay, and generally in some vital part of the structure. Mr. Haines.

This paper relates to zinc chloride, about which the speaker knows nothing; but if the subject of the preservation of timber could be discussed thoroughly and properly and with special reference to creosote as a standard, then conclusions could be reached which would be of great value, not only to railroad companies, but in every other department of industry where timber is being used; and for that reason the speaker wishes that the Society would take up this subject again with one of its committees and bring the state of the art up from 1885 to the present time, as it should be brought, in an authoritative way.

It would be useless to discuss creosote, as the results obtained with that substance are well known from experience in England and in America. The preservation of timber which is properly creosoted is almost indefinite if the proper timber, the proper substance and the proper treatment are used. But this is a question of a cheaper substitute, and the evidence here seems to be that all the cheap substances have been failures, except, perhaps, in Mr. Kruttschnitt's experience since 1894, and, of course, 5 years is too short a time in which to judge of the results. The most optimistic statement in the paper is that the life of ties is 12 years. If considerable is added to the cost of the material by the zinc-chloride treatment, and if then there is some doubt as to whether the life of the tie will be increased to 9 or 12 years, when the untreated timber would have lasted from 4 to 6, a discussion of the subject has not much practical value.

The railroads have not adopted creosoted ties to any great extent, because the users of the timber listen to everybody who comes along. Such persons have all sorts of quack treatments that are ostensibly a good deal cheaper than the real, substantial creosote treatment, which is an expensive one; and the man who is responsible for the annual results does not care to make investments, the benefits of which are to be extended over an indefinite series of years.

The speaker's treatment of timber was not so much with a view to preservation from decay as to preservation from the teredo; and in preserving wood against the teredo, of course a great part of it was exposed to the air and to moisture, in a very hot country where there is much rain. In that way the speaker learned a great deal about the value of the creosote process. In order that the creosoting may be successful the timber and the creosote must be of the right kind and must be subjected to the right kind of treatment.

The creosotes are all hydrocarbons. The lighter parts of so-called creosote are of no value, simply because they are lighter than water, and anything that is lighter than water is soluble therein; but a certain proportion of these lighter parts must be retained, in order to keep the creosote fluid, so that it can be used under pressure. To be

Mr. Haines. of any value creosote cannot be used in any other way than under pressure, and certain conditions must be complied with. The wood must be taken while it is green, and the sap exhausted while the ducts are all open. The creosote must be put in under a suitable pressure until the timber weighs a sufficient amount per foot. Mr. Lowinson has said very properly that a great deal of the value of this process depends on the integrity of the persons who conduct it. There must be some means of inspecting the timber when it is used and of knowing that it is what it is represented to be. With creosote it is entirely practicable to know that what has been paid for has been obtained.

The speaker wishes to call attention to the little progress that has been made in wood preservation, to judge from this paper, since 1885, and, further, to the fact that the subject is a broader one than the mere treatment of cross-tie timber by the chloride of zinc process; and it is to be hoped that this Society will take up the whole subject again and bring up the state of the art from 1885 to the present time. If that can be done the author will have accomplished a great deal by bringing the matter to the attention of the Society.

Mr. Boggs. J. I. Boggs, Assoc. M. Am. Soc. C. E. (by letter).—This paper is both interesting and instructive. In the year 1877 a number of creosoted pole pine ties were put in along the line of the Houston and Texas Central Railroad. During the years 1897 and 1898 the writer had charge of the tracks on the Northern Division of this line, and in the course of his duties had to relay about ten miles of steel, a number of these ties being in the portion so relaid. They were examined closely, and found to be in an excellent state of preservation after twenty years of service, hardly 5% of the total number being rejected as worn out and useless. The most serious objection to them was that the rail cut into the soft wood. This, however, can be entirely avoided by the use of a good, substantial tie-plate, and the writer feels no hesitancy in saying that fully 80% of the original number are still in service.

Various kinds of ties were in use along this line during the years intervening between 1890 and 1899, such as white oak, post oak, cedar, Bois D'Arc, creosoted pole pine and Burnettized sawed pine ties. These latter ties were small, being only 6 ins. x 8 ins. x 8 ft., and the great difficulty, as stated by Mr. Mahl in his report, was that they shattered badly under the heavy traffic to which the road was subjected.

No material record of the life of ties was kept before the latter part of 1897, and in the case of the Burnettized ties the personal observation of the writer was such as to lead him to believe that fully half the ties reported as rotten by the section foremen were shattered instead. He has examined the wood in the ties that had splintered, and the original timber was found to be in an excellent state of preser-

vation in a number of cases, although it was broken and worthless. The writer does not believe that the solution of the problem will be found in the weakening of the zinc chloride, but in the adoption of a better class of material than that heretofore used. The same objection applies to these ties as was stated in the case of the pole pine, *viz.*, that the rail cuts into the soft wood very badly, sometimes as much as 2 to 2½ ins. Six years is about the best life that could be obtained on this road, those placed in 1891 being practically all gone in 1898; but, as before stated, the reports of the section foremen on this subject are entirely unreliable, and can in no measure be depended upon as to the causes for removal. Mr. Boggs.

While the first cost of the creosoted pole pine tie is much greater than the Burnettized tie, a few years' service will readily demonstrate that it is by far the more economical.

B. M. Wagner, Assoc. M. Am. Soc. C. E. (by letter).—During 1890 the writer visited the impregnating plant of the Swiss Nord Ost Bahn and jotted down the following notes which may be of interest. Mr. Wagner.

Zinc chloride is used. The plant consists of two cylinders, each 1.9 m. in diameter and 14 m. long; mixing tanks, each holding about 6 000 l., a 15-H.-P. steam plant and various buildings. This plant is said to have cost 120 000 francs. The cross-ties are run in on rail wagons, shaped to conform to the cylinders, each holding about forty cross-ties.

One hundred liters of zinc chloride, containing about 25% of metallic zinc are put in each of the mixing tanks and the tanks then filled up flush with water. The cost of impregnating one cross-tie is stated to be from 0.45 to 0.48 franc.

The plant is never idle, as the company is overrun with orders from outsiders. The charge is 10 francs per cubic meter of wood impregnated, while the cost, including interest on capital, sinking fund, etc., is only 4.80 to 5.20 francs per cubic meter.

The quantity of zinc chloride used per cross-tie is as follows:

Oak, 8 kls.; larch, 15 kls.; beech, 20 kls.; pine, 25 kls.

The life of ties is as follows:

	Green.	Impregnated.
Oak	15 years.	20–25 years.
Larch	9–10 "	15–17 "
Fir	7– 8 "	12–14 "
Pine	4– 5 "	9–10 "
Beech	2– 3 "	10 "

George W. Noyes, Esq. (by letter).—There are several items which the author has dwelt upon, which are of vital importance to the process. The writer, having had ten years' experience in the operation of the Atchison, Topeka and Santa Fé Railroad Company's timber-treating plant, may be able to explain more fully why results in some cases have not been what might have been expected. Mr. Noyes.

Mr. Noyes. While it is the intention of this company to use nothing but fall and winter-cut ties, it is a fact that many thousand summer-cut ties, in which the fungi had already attacked and partly destroyed the fibers, have been treated, as well as those which were practically worthless at the time of treatment. It is the habit of the native tie choppers to go into the timber in the fall and winter, and fell a sufficient number of trees to fill their sub-contracts, making the ties later on. Many of these trees are allowed to lie from one to two years with the bark on. Then they are made into ties, brought in and received as first-class and treated, when they were really worthless, as the fungoid life had already destroyed the fibers of the wood. To these two classes of ties can be attributed a large percentage of the tie failures, in the past, and those which will occur for some time to come, as it was only last season that the writer succeeded in stopping this practice.

The proper seasoning of timber, both before and after treatment, is of the utmost importance. To this can be attributed, in a great measure, the success of the 1885 ties. On account of the solubility of the zinc salt, it is necessary that all the water injected with the chloride be evaporated before the tie is inserted in the track, otherwise the moisture in the soil at once begins its work of leaching out the zinc salt, and the artificial leathery pellicles formed by the glue and tannin are of no benefit, as they have not had time to dry and harden sufficiently to offer any resistance to outside moisture.

The quality of the zinc chloride has much to do with the success of the operation. The company's first purchases of fused chloride, from 1890 to 1895, were supposed to contain from 3 to 5% of impurities, mostly moisture, but these were, in fact, much greater. In 1894 some chloride was found to contain as high as 2.5% of iron, and its effect is already being realized, as a great many ties treated in this year are entirely gone now. From 1885 to 1890 there were no less than eight different persons in charge of the plant. They were not familiar with the process, and by the time they were the works had closed, and they had sought other employment. This, the writer thinks, was detrimental to the process. It must also be remembered that the first treatment was with a 0.015% solution, injecting only about 0.25 lb. zinc chloride per cubic foot, while the more modern practice is to use nearly a 0.03% solution, injecting about 0.50 lb. of chloride per cubic foot, which should make the ties remain antiseptic for a greater length of time. The following is the record of some 1885 ties treated at these works, and laid in the main line just north of the Kaw River Bridge in North Topeka, Kan.: Of 305 ties put in track, 152 were on a tangent and 153 on a 4° curve. In October, 1895, all of those on the curve were taken out, not so

much on account of rot as wearing under the rail and respiking, which had made their general condition such that the spikes would not hold. Of those on the tangent, all were in on this date, but 22 were to come out, leaving 130 in the track. On October 31st, 1896, there were still 81 in on the tangent, apparently good for several years. The average life of oak ties at this point is 7 years. Mr. Noyes.

As to the Southern Pacific ties: In the fall of 1896 the writer spent some time in examining ties in their track, and, while the zinc chloride and steaming may be the direct cause of their failure, it is, in his opinion, due to the use of superheated steam (as he understands that they used a superheater during the first few years the works were operated), and to a particular species of timber used, as there are several species of timber which will check along the annual rings or radially if exposed to the sun or dried too rapidly by artificial heat. At the company's works, the only kind of timber treated is red spruce, which corresponds to the Oregon fir, and which treats better when green than dry. This phenomenon is no doubt due to the resinous nature of the wood, the resin, on becoming dry, forming an impenetrable barrier which the zinc solution cannot pass.

In the summer of 1896 the company treated (with zinc chloride only), for the Burlington and Missouri River Railway, in Nebraska, two sections of ties from the Big Horn Mountains in Wyoming. The original sections were 16 ft. long, half being retained there and sent to the laboratory at Aurora, Ill. The results of the tests on the treated and untreated timber are shown in Table No. 13.

TABLE No. 13.

Sample No.	Transverse strength.	Deflection.	Crushed under 30 000 lbs.	Spike hold.	Ounces of Zinc Chloride per Cubic Foot.	
					Center.	Edge.
	Pounds.	Inches.	Inches.	Pounds.		
2. Treated	2 766	0.45	0.97	2 017	0.00	12.00
Untreated	2 597	0.48	0.73	1 830	.....	.....
3. Treated	3 125	0.45	0.45	2 600	0.00	27.50
Untreated	3 066	0.54	0.54	2 400	.....	.....
5. New Mexico Burnettized Pine tie	2 975	0.45	0.61	2 775	0.24	6.80
6. New Mexico Wheelhouse Pine tie	2 975	0.38	0.73	2 470	0.62	9.43

Tests of the transverse strength were made on samples 2 x 2 ins. in section, on supports 12 ins. apart. The crushing test was made with samples 6 x 6 ins., by noting the depth to which the flange of a 66-lb. rail penetrated the sample under a load of 30 000 lbs. The figures under "spike hold" show the pull neces-

Mr. Noyes. sary to start a spike which had been driven to a fixed depth. Fine borings from the Wyoming ties were washed in running water for 48 hours, then washed in distilled water and analyzed for zinc chloride. Though they contained 20.8 oz. of zinc chloride per cubic foot before washing they were found to contain none after this treatment.

In the summer of 1896, in order to determine whether the Burnettized or Wellhouse ties retained the more zinc chloride, five ties, which had been treated by the two processes, and which had been in the ground for 6 years, were taken up, and 6-in. blocks cut from each end and from the middle of each tie, making thirty blocks in all. Analyses of ash, as made by Robert W. Hunt & Co., Chicago, showed the average of ten end blocks of the Burnettized ties to contain 0.522% of zinc chloride.

The average of ten end blocks by the Wellhouse process	0.674	per cent.
The average of five middle blocks by the Burnettizing process	0.496	"
The average of five middle blocks by the Wellhouse process	0.525	"
The average of the entire fifteen Burnettized blocks	0.513	"
The average of the entire fifteen Wellhouse blocks	0.624	"

The general conditions as to drainage, soil, traffic, etc., were the same in both cases.

While this test showed more zinc chloride retained in the ties treated by the Wellhouse process, yet there is one point which has never been touched upon, *i. e.*, the organic decomposition of the glue when not fixed by the tannic acid. This is a very important point, and should be a strong argument in favor of the three solutions as now used by the Chicago Tie Preserving Company.

The fact that it takes a great deal longer for the Wellhouse ties to dry after treatment, proves that the treatment does, to a certain extent, prevent the passage of moisture into the wood.

Mr. Wallace. JOHN FINDLEY WALLACE, M. Am. Soc. C. E. (by letter).—This paper discusses a question which is becoming yearly more and more important to the railroad companies of the United States. The annual increase in the amount of ties used for renewals and the steady diminution of the timber supply are gradually attracting the attention of railroad owners and managers to the tie question.

There seems to be little doubt that, from now on, more and more attention will have to be given to preservation methods, but this is only a step toward the solution of the problem, which must undoubtedly come in the use of some substitute for the wooden tie, and which, it would seem, must eventually compel the use of a metallic tie.

Mr. Wallace.

An investigation which the writer has just completed shows that the average life of the untreated white oak tie in northern Iowa is $11\frac{1}{2}$ years, in Illinois 10 years, and in Kentucky and Tennessee and the high ground in Mississippi $7\frac{1}{2}$ years. These figures apply to winter-cut white oak ties grown on the high lands of Kentucky, Tennessee and Mississippi.

Black and red cypress ties, used in the low lands of Mississippi and Louisiana, have an average life of 10 years, and this life can be increased several years by the use of tie-plates, as these ties wear out before they decay.

These figures are taken from the actual records of ties used during the 10 years ending 1898, and the writer considers them reliable for the general territory of which the several districts mentioned form a part.

One difficulty met by railroad managers, in considering tie-preservation methods, is the want of accurate and reliable information as to the actual increase in life due to the various processes of treatment applied to the different kinds of material. Generally, the close-grained woods, similar to white oak, are not susceptible of economical treatment. Generally speaking, the woods which absorb a sufficient amount of the preserving material are coarse of grain, and it is usually found necessary to use tie-plates with them, at an additional expense of 10 to 15 cents per tie, in order to get the full benefit of the preserving process and prevent the tie from wearing out before it decays.

When new processes are brought before railroad managers, actual facts are necessary for the passing of proper judgment on what is the best and most economical thing to do, under the circumstances. It is very difficult to prevail on railroad managers to enter into any line of expenditure which is more or less experimental. Many railroad managers have given more or less general attention to the question of treated ties, but have, so far, not felt justified in taking practical steps toward the general use of any of the existing processes, on account of the expense and uncertainty as to results, although there is little question but that the necessities of the near future will force action in this direction.

Mr. Tratman.

E. E. Russell Tratman, Assoc. M. Am. Soc. C. E. (by letter).— One of the most important points of this paper is the very forcible way in which it shows that no reliable records exist or are kept in the United States as to the life of railway ties, from which averages of life and cost of different kinds of timber can be computed. Without such records, the calculations and estimates of comparative durability and economy of different ties and different preservative processes, are really little more than guess-work (except for individual roads), as there is no real foundation upon which to base any comparisons. The few railways which keep useful records have generally such diverse forms and methods, that it is difficult to make comparisons between them.

Mr. Tratman. The ties are a very important consideration, in their aggregate cost, the labor of renewal, the disadvantages of frequent renewals (from traffic and economic points of view), and the consumption of forest timber which these ties represent. In view of all this, it is most desirable, even at this late day, to endeavor to have some uniform system adopted by which records may be kept which shall be complete, comparable, and free from the influence of the personal equation which makes present statistics so unreliable as a basis for any general conclusions. It is, of course, difficult to get a system of this kind introduced and properly followed out, but the only way is to make a beginning of some kind, and then gradually develop it on lines carefully thought out.

It seems to the writer, that the keeping of these records upon some uniform and carefully formulated system, is a duty that might be properly imposed upon the division roadmasters, or officers of similar position. The section foremen should be required to keep very careful record of the number and cause of tie renewals (decay or rail-cutting). The roadmasters should then, from this information and their own knowledge and observation, prepare the broader records. Some means of supervision should also be devised, to ensure that the inspection and renewal of ties and the keeping of the records are done carefully and not in a mere hap-hazard or perfunctory manner.

The following list shows some of the items which should be included, and Table No. 14 is a sample of the suggested form of record. With such complete and uniform records from a number of railways for a term of years, and with the records of the operations of preservative processes and their cost, some broader and more reliable information could be compiled than is possible with existing statistics. This information would undoubtedly lead to greater care in the use of ties.

1. Miles of single main track on the division.
2. Approximate tonnage carried over the division during the year.
3. Kinds of timber used and the percentage of each.
4. Number of ties per mile (according to the standard spacing or standard number per rail length).
5. Preservative treatment of the ties.
6. Number of ties of each kind renewed.
7. Cost of ties of each kind renewed.
8. Date when ties removed were laid.
9. Place where ties of each kind were obtained.
10. Tie-renewals per mile of track.
11. Kind of ballast in which ties were laid.
12. Weight of rail.
13. Are tie-plates used?
14. General condition of track; including condition of rails, and road bed, surface, line, etc.

TABLE No. 14.—Tie Record, Smithville Division, A. B. & C. Ry. Mr. Tratman.

1	50 miles.		
2	....tons.		
3	Oak, 50%.	Pine, 25%.	Hemlock, 25%.
4	2 464.	2 816.	2 816.
5	None.	10% zinc chloride. 90% none.	10% creosote. 10% zinc chloride. 80% none.
6	5 000.	3 125.	3 750.
7	55 cts.	50 cts.	20 cts.
8	2 000 in 18.... 3 000 in 18	2 000 in 18.... 1 125 in 18....	3 000 in 18.... 1 750 in 18....
9	Pennsylvania.	Georgia.	Pennsylvania.
10	200.	250.	300.
11	Stone.	Screened gravel.	Common gravel.
12	80 lbs.	60 and 70 lbs.	60 and 70 lbs.
13	No.	At joints.	On all ties.
14	Good.	Good.	Rails worn, and surface poor.

C. Stanley Churton, Esq. (by letter).—On reading Mr. Curtis' able and exhaustive paper one is struck by the extremely short life of the ties. This, however, is partly explained when he states that "they perish by the mechanical destruction of the fibers under the rail," and also by the cutting out of the fibers in the process of respiking. Further on it is shown that ties can be protected by plugging the spike holes, and the writer would suggest that before insertion the plugs be dipped, either in liquid creosote, or a solution of chloride of zinc, or sulphate of copper. Mr. Churton.

The whole question of preserving timber by chloride of zinc seems hopelessly mixed. For instance, Mr. Curtis, in remarking on progress since 1885, writes: "Three plants treating timber with zinc chloride were in operation in the United States, each working with a different process," and further alludes to "the method adopted to prevent the impairment of the preservative treatment by the washing out of the zinc salt, which is quite soluble in water."

The paper does not state how one process differed from the others, but the writer thinks that Mr. Curtis is wrong in his statement concerning the washing out of zinc salt. The fact is—and it is an important fact, and one not to be forgotten—that chloride of zinc acts on vegetable fiber the same as iron acts on cotton or linen, forming what is known as iron-mould. The cotton can take up only a certain quantity of iron, but this becomes incorporated with it, and no amount of washing or boiling can take it out; there is a chemical combination between the iron and the linen. An excess of iron would rot and destroy the linen, and the excess which could not be combined with the linen could be washed away.

The fiber of the wood can enter into combination with the zinc up to a certain amount, but more than that would be worse than useless,

Mr. Churton. in fact, would actually defeat its own object and hinder rather than assist the longevity of the timber.

Again, chloride of zinc attracts water from the atmosphere, therefore an excess of chloride would keep the ties perpetually wet; in fact they would never become perfectly dry; consequently, decomposition of the fiber under the rail would be hastened. It is admitted on page 324, that on the Southern Pacific Railroad, the quantity of chloride used was too great.

In some parts of Mr. Curtis' paper, the process is described as being performed under pressure, at others in a "vat," when the solution was raised to a great heat, 250° to 300°. The writer's opinion is that the life of the wood is in the sap; destroy the sap and the life is destroyed. The best way to preserve timber for railway use is to let it be well seasoned before being placed under pressure. The actual drying, under favorable circumstances, does not take very long after 10 lbs. of solution of zinc chloride of 1.025 specific gravity has been forced into it; when dry it is harder, denser and more durable. The use of the word "vat" gives one the impression that in some places ties have been preserved by being macerated in a hot solution of zinc chloride; and that great heat has been used. The timber has, in fact, been boiled in a preservative solution. The object of applying heat—which is not absolutely necessary when timber is treated under pressure—is to expand the pores and to hasten the absorption of the preservative, and the application should be continued only as long as is necessary to absorb the quantity of solution required. To destroy the sap by excessive heat, in fact, by boiling it out, is, in the writer's opinion, a positive mistake.

It cannot be too strongly insisted upon that many theorists, faddists and experimentalists have, from time to time, invented processes for the preservation of timber, which have only had the effect of destroying its vitality.

About twenty-five years ago a process unknown to the writer was much advocated, but after a little time the wood, some specimens of which were shown ten years ago, broke like carrots. Timber, like wine, requires careful and skilful handling, and those whose duty it is to treat it with chemicals should understand thoroughly the nature of the chemicals used.

The late Sir William Burnett, the inventor of Burnettizing, was a medical director in the navy, and was cognizant of the therapeutical effects of chloride of zinc on vegetable fiber. His directions were that one part of liquid chloride of zinc having a specific gravity of 1.600 should be mixed with 40 parts of water, and experience has proved that this is the best strength for treating timber, and most conducive to its longevity.

Mr. Van Vryberghe de Coningh* (by letter).—On the Dutch State Railroad wooden ties have been used exclusively since 1889, as steel ties did not give satisfaction. Baltic pine has been used on tangents and curves of greater radius than 1 000 ms., and Dutch oak or beech for curves of less radius and for switches. Dutch oak is preferred to Baltic oak because having grown more slowly it is consequently tougher, though less showy. Up to the present time no difference has been observed between inland and foreign beech. The ties are from 2.60 to 2.70 ms. in length. Their cross-section is as shown in Fig. 1.† Mr. de Coningh.

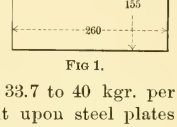

Fig 1.

The rails are of the T-type, weighing from 33.7 to 40 kgr. per meter, and are not fixed directly on the tie, but upon steel plates (shown in Fig. 2), and these, at the same time, give the desired inclination to the track. The length of these plates is 200 mm. The rail and tie-plate are fixed by spikes.

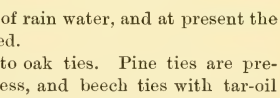

Fig. 2.

Formerly the ties were planed only at the places to be occupied by the plates, by making incisions with a saw and planing between, but these incisions caused the infiltration of rain water, and at present the whole upper surface of the tie is planed.

No preservative process is applied to oak ties. Pine ties are preserved by the modified Rütgers process, and beech ties with tar-oil (creosote). In the future, when the installations have been modified, the Rütgers process will also be applied to the oak ties.

All timber is cut in the winter, and great care is taken to have the ties thoroughly air-dried before they are treated, because, in that condition, the wood absorbs the preserving liquids more readily.

As navigation is closed by the ice during winter and spring, the Baltic pine is shipped to the Netherlands between June and December, and is, therefore, generally piled up in the Netherlands during one winter. Only in exceptional cases, and when there is great need for it, lumber shipped in early summer is put through the process in the fall.

Beech shipped in the spring is used in the summer. It is easily impregnated and is then very durable, but if it is not impregnated it decays very soon. Therefore, great care must be taken with it before it is impregnated, for, if it is piled up carelessly, it decays readily.

The modification of the ordinary Rütgers process concerns only the composition of the impregnating liquid, which is 5° Beaumé (4% $Zn Cl^2$) instead of 3° Beaumé (2% $Zn Cl^2$) as ordinarily used. To 30 kgrs. of the mixture (the dose for one pine tie) is added 5 kgrs. of tar-oil, instead of 2 kgrs.

* *Ingenieur der Staatsspoorwegen*, Utrecht, Holland.

† The dimensions on Figs. 1 and 2 are in millimeters.

Mr. de Coningh. Up to the present time experience has not shown that by augmenting the quantity of zinc chloride the fibers of the wood are attacked, or that the wood becomes brittle. Great care is taken, however, to keep the zinc chloride absolutely free of acid.

The quality of the creosote is fixed by prescription and regularly controlled. The process is wholly that of Burnett, applied by Rütgers: steaming, air-vacuum and impregnating under pressure, the time for each part of the process depending on the degree of moisture in the wood.

Until the impregnated ties are to be used in the track, great care is taken to keep them piled up close together; just contrary to the method in which they are piled before impregnating, which is very loose.

There are no statistics yet as to the life of impregnated ties.

The cost, during the last 6 years was as follows:

Pine ties.....	1.30 to 2 guilders (52 to 80 cents).
Beech "	± 2.70 guilders ($1.08).
Oak "	28 to 38 guilders per cubic meter.

In 1898 the cost per tie was as follows:

Pine.....	0.46 guilder (18½ cents).
Beech.....	0.72 " (29 ").

In these prices are included, respectively, 0.30 and 0.58 guilder (12 and 23 cents) for the liquids (the prices of these varying yearly), and 0.07 guilder (2¾ cents) for labor. The remainder makes up the expenses for engineers, coal, repairs to machinery, and supervision.

Mr. Rowe. Samuel M. Rowe, M. Am. Soc. C. E. (by letter).—These researches by Mr. Curtis certainly have much value, not, perhaps, for any definite conclusions adduced, or that can be adduced, so much as for giving a *résumé* of the question of timber preserving from the time of the report of the Committee of this Society in 1885 to the present time. That this article is resulting in an awakening to the importance of this matter seems to justify its preparation and the great amount of labor and care required, and it is to be hoped that something more than thanks will be accorded.

The pressing needs of the day for cross-ties to supply the waste on existing railroads and for new lines has called attention sharply to the necessity of some method of preventing the rapid depletion of the available stock of timber, particularly as few American railroads are yet prepared to resort to metal ties.

The discussion of this matter, both in this case and generally, is somewhat confusing, but probably pertinent and worthy of consideration. To extricate ourselves from this, however, and to place ourselves on something like solid ground, we are inclined to fall back on experience and give some of the conclusions derived therefrom.

Mr. Rowe.

It came into the province of the writer in 1885, to be placed in charge of the works then just completed at Las Vegas, N. Mex., a position in which it was imperative to make a close study of the philosophy, the practical workings, the methods used, the nature of the chemicals, as well as a thorough analysis of the results attained. The zinc-tannin or Wellhouse patent process was to be used, and in this study the aid of O. Chanute, M. Am. Soc. C. E., and the late Joseph P. Card, Assoc. Am. Soc. C. E., was availed of; they being owners of the then existing patents on the process. It is unnecessary to say that the conclusions there reached were and are still of much value, no occasion for revision to any extent having arisen.

It is a pleasure to read the contribution of Mr. Noyes, in reference to these works, and it is very desirable that more such men enter the field to add to the number of careful observers.

It is not deemed necessary or desirable to go deeply into this discussion at the present time, and the writer will confine his remarks to a general survey of the practical points of the situation and to the importance of educating practical operators, of whom there are too few.

We should not place particular stress on the process used, whether it be the Burnett, the zinc-tannin or the creosote process, the appliances and methods being quite similar, but on correct and efficacious methods, the main point being to get the absorption of a proper amount of chemicals, or rather to prepare the timber for it. To do this, a thorough knowledge of the absorptive qualities of the timber to be treated, as to density, dryness, etc., must be had, after which judgment and discretion, with some degree of practical experience, will be the best guides.

In regard to the question of securing any desired degree of absorption, it depends on the density of the timber; but as to the effectiveness of the process used (steaming or the vacuum), there is little question. By natural absorption, that is, by soaking or immersing in water, would require, under the most favorable circumstances, some thirty days to absorb as much as will be injected by this method in a few hours (five or six altogether).

It should be remembered, too, that the amount of chemicals, or antiseptics, has not all to do with the results, neither can a superabundance of the chemical be relied upon to make amends for lack of judgment in respect to other parts of the treatment. The overplus above that which long experience and trial in the track has shown to be necessary is worse than money thrown away. The saying in the "Hoosier Schoolmaster," "While yer gittin git a plenty," will hardly hold in this case.

The question of choosing open-grained timber is to some extent misleading, as the writer's observations for the last 15 years lead him

Mr. Rowe. to believe that the expression "the better the timber, the better the tie," is in the main true, the fact remaining that many of the timbers now deemed worthless for this purpose, may be made lasting and serviceable.

The mention of small ties, 6 x 8 ins., deserves notice. A sawed tie less than 7 by 9 ins can on no rational grounds be defended.

The degree of drying which a tie should have should be sufficient to relieve the wood of at least half its water; the time required to do this depending on climate, moisture and other conditions which vary in different parts of the country. There may be exceptions as to this with timbers which have not yet come under the writer's notice.

If the writer were permitted, he would say to the railroad manager, that in many cases such a plant, equipped and operated, would prove of as much advantage as a repair shop, and should have the same attention and care; the manager of the works being educated in this particular line and equally as well paid as the master-mechanic of his repair shop. No man can be prepared for this without close contact with the practical part of the work and a close study of its operation.

As an example of this, one of the largest railroad systems in the United States, put aside all theories and speculations as to particular methods and took the tangible results shown on the financial sheets of cost of maintenance, viz., a reduction of renewals from over 20% in 1885, to less than 5% in 1897-98.

Mr. Eayrs. N. W. Eayrs, M. Am. Soc. C. E. (by letter).—The author has presented a deal of valuable information on the subject of preservation of railroad ties, but it is to be regretted that the paper was not extended to include ties and timber used in bridges and viaducts. The cost of renewing ties on such structures and under traffic is very much greater than that of tie renewals on ballast, and every year added to the life of such ties is more valuable than in the case of common ties.

The writer's experience in recent years has been chiefly in the maintenance of bridges, viaducts and elevated structures, and he has been surprised to find that so little effort has been made to protect bridge ties and floors from the action of the weather. For a number of years past it has been the writer's practice to paint bridge ties, and the result has been so satisfactory that he is inclined to doubt the advisability of treating such ties by any other preservative process. One great objection to using the ordinary processes for preserving such timber is, that, as universally admitted, timber should not be cut after treatment; it was suggested to the writer by the management of one preserving works that the ties should even be bored for spikes before treatment. The expense of such rehandling, and the extra freight charges, would be prohibitive.

Mr. Eayrs.

The writer's experience on the St. Louis Bridge is an instance of the value of painting ties. The railway floor of this bridge was changed to a tie floor in 1888; the ties used were 7 x 7-in., long-leaf yellow pine, carefully selected. All daps were painted before the ties were placed, and also all surfaces where wood came in contact with wood. After the floor was finished the ties were given two coats of red oxide of iron and linseed oil. These ties are in perfectly sound condition to-day. Unfortunately, tie-plates were not used when the ties were first laid, and it is now necessary to renew them on account of "spike killing" and wear; otherwise the ties would be good for 5 years longer, if not more.

In 1897, the writer renewed the entire floor of the elevated portion of the St. Louis Merchants Bridge Terminal Railway. This floor was laid in 1891 with oak ties, unpainted and untreated. Six years afterward more than 50% of the ties were so badly decayed that nothing was left but a shell. The traffic over this structure had been very light up to 1896, and, consequently, the ties had suffered no mechanical injury. The timber used in relaying was long-leaf yellow pine, and was painted with one coat of mineral paint after framing, and one coat after being placed in the track. The cost of painting the ties, two coats, averaged $11\frac{3}{4}$ cents per tie. Tie-plates were used on this structure, and, judging from the experience on the St. Louis Bridge, these ties ought to last 15 years. About 75 ties were treated by dipping into a bath of hot Woodiline, and about 150 others were painted with Woodiline applied hot. The dipped ties were allowed to remain in the bath ten minutes; the amount absorbed was $22\frac{1}{2}$ galls. per 1 000 ft. B. M., or about $1\frac{1}{3}$ galls. per tie. The cost of the work was 52 cents per tie for material, and about $1\frac{3}{4}$ cents per tie for labor in handling, exclusive of the cost of the tank, steam connection and fuel.

Taking into consideration the cost of the process, the uncertainty of getting the proper amount of preservative in the timber, and the necessity of treating after the framing has been done, it is the writer's opinion that for bridge ties there is very little more economy in any of the preserving processes now in use than in painting.

Mr. Selby.

O. E. Selby, Jun. Am. Soc. C. E. (by letter).—Table No. 15 and the diagram, Fig. 3, were prepared by the writer during an investigation of the question of timber preservation for the railroad system with which he is connected, for the purpose of arriving at a rational and ready means of comparing the economy of different methods of preservation.

The diagram, omitting the four-sided figures lettered *A*, *B*, *C*, etc., is plotted from Table No. 1, page 195, of Bulletin No. 9, United States Department of Agriculture, Division of Forestry. The abscissas represent the cost in cents of each renewal, and the diagonal lines represent the intervals between renewals. The ordinates show the annual

TABLE No. 15.

	Kind of Tie and Treatment.	Cost of.			Total cost in track.	Probable life. Years.	Annual charge. Cents.	Life Necessary to Equal White Oak Tie.	
		Tie delivered.	Treatment.	Putting in track.				At 38c.	At 50c.
A....	First quality white oak, untreated. Present prices...	35 to 38	.....	12 to 15	47 to 53	8 to 10	6.1 to 8.2	.....	.....
B.....	" " " " " Future " ...	50 " 55	.....	"	62 " 70	8 " 10	8.0 " 10.8	.....	.....
C....	Inferior oaks, untreated.....	30 " 34	.....	"	42 " 49	4 " 5	9.7 " 13.8	.....	.....
D....	" " Burnettized.....	30 " 34	16 to 20	"	58 " 69	10 " 14	5.9 " 8.9	$11\frac{1}{2}$	9
E....	Hemlock, gum, etc., untreated.....	28 " 32	.....	"	40 " 47	3 " 4	11.3 " 16.7	.....	.....
F....	" " " Burnettized.....	28 " 32	16 to 20	"	56 " 67	9 " 12	6.3 " 9.4	11	$8\frac{1}{2}$
G....	" " " " and tie-plates.....	28 " 32	36 " 40	"	76 " 87	11 " 14	7.7 " 10.5	$16\frac{1}{2}$	12
H....	Beech, untreated.....	30 " 32	.....	"	42 " 47	3 " 4	11.8 " 16.7	.....	.....
J.....	" Burnettized.....	30 " 32	16 to 20	"	58 " 67	12 " 15	5.6 " 7.5	$11\frac{1}{2}$	9
K....	" minimum life, Burnettized.....	30 " 32	16 " 20	"	58 " 67	9 " 10	7.5 " 9.4	.....	.....
L....	" " " and tie-plates.....	30 " 32	36 " 40	"	78 " 87	14 " 16	7.2 " 8.8	17	12
M....	Chestnut, untreated.....	30 " 34	.....	"	42 " 49	5 " 6	8.3 " 11.3	.....	.....
N....	" " and tie-plates.....	30 " 34	20	"	62 " 69	12 " 14	6.3 " 7.7	12	9
O....	Cedar, "	35	.....	"	47 " 50	5 " 6	9.3 " 11.5	.....	.....
P....	" " and tie-plates.....	"	20	"	67 " 70	12 " 14	6.8 " 7.8	13	10
Q....	Inferior yellow pine, untreated.....	45	.....	"	57 " 60	4 " 6	11.1 " 16.9	.....	.....
R.....	" " " creosoted.....	"	15	"	102 " 105	20 " 22	7.7 " 8.4	23	$17\frac{1}{2}$
S.....	" " " vulcanized.....	"	23 to 30	"	80 " 90	15 " 16	7.4 " 8.7	16 to 18	12 to 13
T....	" " " " and tie-plates.....	"	43	"	100 " 103	16 " 18	8.5 " 9.5	24	17

Mr. Selby.

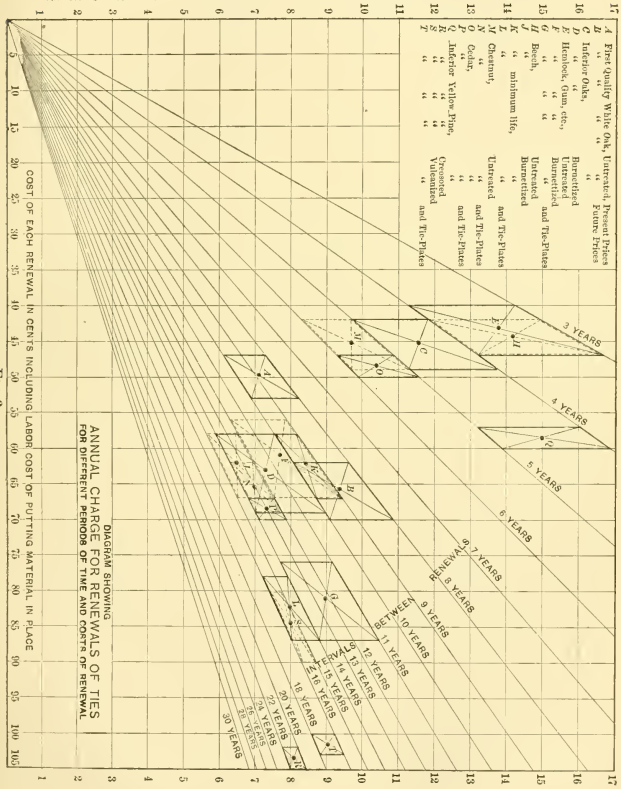

Fig. 8.

Mr. Selby. charge in cents, based on 5%, compound interest, necessary to liquidate the succeeding renewals at the intervals shown. In other words, it is the amount which it would be necessary to set aside each year and invest at 5%, compound interest, in order to equal, in the end, the cost of all renewals reckoned at the same rate of interest.

The diagram shows graphically and exactly the comparative ultimate economy of various methods involving various lengths of life and initial costs. The only approximation involved is in the assumed lengths of life and assumed costs. Comparisons can be made for structural timber by taking the costs and annual charges in dollars per 1 000 ft. B. M., instead of in cents per tie.

While it is known that most railroads can now obtain money at less than 5%, it is believed that the addition of general expenses, taxes, costs of financing, uncertainties in distribution of expenses, etc., makes a 5% basis of comparison a proper one, especially as it works to the disadvantage of the longer-lived ties and processes, as to the durability of which there is necessarily much uncertainty.

The table was prepared to include the principal kinds of ties and methods of treatment available on a large scale on the system in question. There are several species of wood, such as catalpa, sassafras, black walnut, etc., which are known to be very durable and valuable for cross-ties, but which are not available in large enough quantities to have much effect on the tie supply of a large system, and they were not included in the comparison.

The basis of comparison is the first-class tie, of white oak, burr oak or post oak, filling the present specifications of the system, costing 35 cents to 38 cents each, and lasting in the track from 8 to 10 years, under reasonably favorable conditions. In view of the increasing scarcity of such first-quality oak, and rapidly advancing prices, a comparison is also made on a basis of the probable early future cost of 50 cents each. The assumptions in the table, as to cost of ties, treatment, putting in track and probable life, are based on reports from division engineers, and information from other sources, including Mr. Curtis' paper, applicable to the middle western territory covered by the system. Limits were used rather than averages so as to show the possible range covered by each process.

The column headed "Total Cost in Track" is made up by adding together the cost of the tie delivered, the cost of treatment, if any, and the labor cost involved in removing the old tie and putting in the new one. It is important that this item of cost of putting in track be included in all comparisons, as it is certainly an expense involved in each renewal, and its omission gives an unfavorable advantage to the short-lived ties and processes. The column headed "Annual Charge" is taken from the diagram for the preceding data of total cost in track and probable life. The last two columns of the table show the length

of life which must be realized by the different processes at the assumed costs in order to equal in economy a white oak tie, first, at present prices ; second, at future prices. This was obtained from the diagram by following out the horizontal line representing the average annual charge for the oak tie to its intersection with the vertical line representing the average cost of the treated tie. The year lying nearest this intersection is the required life. Mr. Selby.

The use of metal tie-plates, with treated and untreated ties, was considered, together with other methods of preservation, and the results show it to be a most promising method with soft wood ties. The cost of tie-plates was assumed at 10 cents each, including the cost of their transportation to the site and putting them in. This may seem high for small plates, but it is likely to be realized with the advancing prices of steel.

Two assumptions were made for Burnettized beech ties, the first based on their claimed and probable life of 12 to 15 years, and the second on a life of 9 to 10 years, which seems a certain minimum, in view of experience with the process and of improvements now being tried. The range of cost of vulcanizing was made large to cover both the present cost and the future reduced cost of the process if it proves successful and works are established in the West. In view of developments in the tie market, since the table was made, the price of cedar ties was probably taken too low.

The data in the table have been plotted on the diagram, each kind of tie being represented by a four-sided figure, within the limits of which will be found the actual annual charge, the intersection of the diagonals locating the average. Those plotting nearest to the base line show the lowest annual charge, and hence the greatest economy.

The one which plots lowest is the beech Burnettized, but if its life is proven to be only 9 or 10 years it plots much higher, but is still lower than the white oak tie at 50 cents. The chestnut tie with plates, the cedar tie with plates, and the inferior oaks Burnettized, all show greater economy than the white oak tie at 38 cents delivered. All the processes of treatment plotted show greater ultimate economy than a white oak tie which costs 50 cents delivered and lasts only 8 to 10 years, indicating that the time is rapidly approaching when some form of preservation will be imperative.

On the face of the diagram the best showing is made by the Burnettizing process, and by untreated, long-lived, soft wood ties with plates.

The flat inclination of the lines representing the longer periods of years shows that the amount we are justified in spending for a tie increases rapidly with the life realized; that is, the annual charge increases more slowly in comparison with the first cost for the long-lived ties than it does for the short-lived ties.

Mr. Selby. The cost and probable life in any individual case may easily fall outside the limits plotted, but in any case it is easy to follow out the horizontal line corresponding to the cost in track and probable life of ties in use to find the life which it will be necessary to realize from a process of given cost, or the amount which it is justifiable to pay out to secure a tie of a given life.

The writer's investigation resulted in a recommendation for the purchase of a considerable number of inferior oak ties and their treatment by Burnettizing, and the purchase of a like number of cedar and chestnut ties and their use with tie-plates; the number recommended being large enough to be considered experimental on a large scale.

There is much promise in a modified Burnettizing process, now being investigated by a leading tie-preserving company, consisting of the injection of both creosote and zinc chloride. This is likely to prove durable, but may be too expensive. The writer agrees with others who have taken part in the discussion that much of the hesitancy of the managements in authorizing the use of treated ties is on account of the uncertainty which exists as to the actual life, under varying conditions, both of ties now in use and of the treated ties.

Certainty, in this direction, is only to be attained by careful records of all ties in the track, showing length of service and actual cause of removal. Such records would be of value in determining, not only the necessity for and the best processes of treatment, but in indicating the value of improvements in any given process.

Mr. Chanute. O. CHANUTE, Past-President, Am. Soc. C. E. (by letter).—This paper chances to be most timely, for its publication concurs with one of those swift advances in prices which take place when it is suddenly discovered that the supply has been overtaken by the demand. One year ago ties were selling about as cheaply as they had been for 10 years past. In Chicago, for instance, white oak ties were quoted at 45 cents and cedar ties at 36 cents, while now the price of white oak is 60 to 65 cents, and of cedar 50 cents, with talk of further advance. Railroad men, therefore, are considering now the use and treatment of the cheaper and more perishable kinds of wood, which are still abundant.

There is no longer any question in the mind of the writer that chemical treatment, applied thoroughly and skilfully, will more than double the life of the inferior woods, and make it equal or superior to that of the more durable woods laid in their natural state. All the European experience has proved this, and in spite of some failures, due to ignorance of all the conditions of success, American experience is seen to confirm that of the Europeans.

The writer drifted into this business by chance some 14 years ago. He sees now that he made some mistakes in the beginning, and had much to learn concerning the idiosyncracies of various woods, so that his experience may be of value to those who may carry on such

work hereafter. His first mistake consisted in the apprehension that strong solutions would make the ties unduly brittle, and in believing that solutions of zinc chloride of 2° Beaumé, or less, would penetrate farther into the wood than stronger and less limpid solutions. Mr. Chanute.

In 1879, Mr. B. F. Bischoff, then in charge of the tie-preserving works of the "Empress Elizabeth Railway," of Austria, read an elaborate paper before the Austrian Railroad Club, in which, after describing the then state of the art, he gave an account of some experiments which he had made, and which went to show that zinc-chloride solutions of 1.7° Beaumé could be forced into the wood further, and that they protected the ties better, than solutions of 3° Beaumé. He instanced, as proof, the results cited by Mr. Funk, in a paper which has remained a classic, concerning the life of ties treated with solutions of 2° Beaumé, and he stated that the "Empress Elizabeth Railway" had changed over from the use of zinc-chloride solutions of 3° Beaumé to those of 1.7° Beaumé. This practice, which seemed to promise some economy, was followed by many German railways, but the results were not found satisfactory. It was ascertained that these railways did not do as good work as others, which used stronger solutions, and the German specifications of 1898 and 1899 require the use of solutions of 3.5° Beaumé, and the injection of 0.50 to 0.55 lbs. of dry zinc chloride per cubic foot of wood.

In consequence of the fact that American ties get less seasoning than is usual in Germany, the writer began treating ties at Chicago in 1886 with solutions of 3.5°Beaumé, at 80° Fahr., and he has gradually increased the strength to 5° Beaumé, having in the meantime learned by experiment how to inject these strong solutions without making the tie brittle. Not one of them has broken in the track. He is now injecting two and a half times as much chloride of zinc as he did at the beginning, and has every reason to believe that the results, with ties treated since 1896, will be entirely satisfactory.

His second mistake consisted in assuming that there was no great difference in the quantity of solution absorbed by each individual tie in a run, and that they might be treated with success in the order of their arrival. Unfortunately, ties arrive in Chicago by vessels in varying condition; some have been cut longer and are better seasoned than others, while particular cargoes, or parts of cargoes, have been rafted out to the vessels, and are saturated with water. It was believed at first that these varying conditions could be overcome simply by changing the strength of the solution, from day to day, to correspond, and that satisfying results would follow. This proved to be an error, and now refractory ties are unloaded upon the ground to season; some 70 000 having been so treated in 1898, and some 120 000 in 1899. Moreover, hundreds of experiments with individual ties, made by weighing them just before and after treatment, have established the

Mr. Chanute. fact that there are surprising differences in the receptivity of various kinds of wood, and of various ties cut at the same time and from the same forest. Some will absorb three times as much as others, and, in extreme cases, when both seasoned and unseasoned ties are tested, the differences are as 8 to 1. The conclusion drawn was, that, in order to inject the minimum of zinc chloride guaranteed in every tie, it was necessary to inject on an average about three or four times this quantity, and this is now being done.

This difference between individual ties was found to be greatest with tamarack, probably in consequence of the varying amount of resin contained therein, and, after many experiments, the treatment of ties of this class was reluctantly abandoned.

Thus it is seen, that, simple as the process appears, theoretical knowledge alone is not sufficient to insure doing good work. Practical experience is required constantly, and this is acquired best with a small experimental plant, with which various methods may be tested without interfering with the regular runs.

It is unfortunate that various changes in the personnel of the maintenance-of-way departments of some of the railroads first adopting tie-preserving methods in this country should have produced discontinuity in the records, but these are sufficient, as Mr. Curtis has shown, to prove that inferior woods are made more durable than white oak, and we may expect still better results now that experience has taught us how to do the work better.

The writer is satisfied that his present practice of injecting from 0.50 to 0.55 lbs. of zinc chloride per cubic foot of wood is an improvement over his earlier methods. Also, that the injection of three separate solutions instead of two is a decided improvement. He believes the zinc-tannin process, thus applied, to be better than the zinc process alone, and that experience in regions of abundant rainfall will prove this to be a fact. Burnettizing, as we know, has been abandoned in England, while it is a success in Germany, probably in consequence of the difference in rainfall in those two countries.

In the hope of learning still more about the anomalies of various woods, and the best methods of treating them, the writer is now about to visit Europe, where far more experience has been accumulated than in America, and he regrets that the present publication of this valuable paper will prevent this discussion from containing the results of his pending tour of observation.

Mr. Curtis. Walter W. Curtis, M. Am. Soc. C. E. (by letter).—When the author undertook the investigation of the treatment of railroad ties, he was desirous of making it complete, if possible; and for this purpose applied for information to the manufacturers of all preserving materials, to all the treating works in this country and to all the railroads which, as far as he could ascertain, had ever experimented with

treated ties. The vulcanizing process was the only one, known to him, not investigated. Mr. Curtis.

He was favored with replies to most of his inquiries, and followed up all the avenues of investigation opened. The resulting information regarding other preservatives than zinc chloride was so slight as to be practically valueless, and he concluded to omit all mention of them.

There is no question of the superiority of creosoting over any other treatment, but it would be rather difficult to prove it by American records. On account of the cost of the treatment, most of the creosoted material has been used for other purposes than ties; and the object sought has been not so much protection from decay as from destruction by marine life.

Where creosoted ties have been used, generally no record of them has been kept, and no accurate life for them can be computed. No figures comparable, either as to accuracy or number considered, with those given in the paper for Burnettized ties on the Southern Pacific or the Atchison, Topeka and Santa Fé roads, are obtainable.

The author does not desire to be understood as condemning any of the processes now in use. He simply could not find sufficient evidence to justify him in presenting them.

It is unfortunate that it is impossible to give exact data covering all the ties treated with zinc chloride; but it seems unwise and unscientific to belittle what we have because it is not more. The material question is: Is our information sufficient to enable us to draw reasonably correct conclusions—not, is it perfect? The statement made by one gentleman, that we are no further ahead in our knowledge of the subject in 1899 than we were in 1885, in view of the records of the two roads above mentioned, omitting all others, is absurd.

Mr. Lowinson speaks of an unfavorable experience with Burnettized ties on the Erie Railroad. This statement has been the subject of the same critical examination given to all in the original investigation; and it is proven to be entirely in error. The Erie Railroad has made no experiments with Burnettized ties, except as specified in the report of the Committee of 1885, and as quoted in this paper. The author is advised by Mr. Lowinson that his statement was incorrect; due either to his misunderstanding information given him or to misinformation. To the author it seems probable that it was due to a confusion of processes.

Replying further to Mr. Lowinson's discussion, the distribution of the chloride throughout a tie is quite uniform. The analysis of a tie which had been in service for 11 years on the Union Pacific Railroad, as given on page 311, showed the same amount of the preservative at the center as at the ends of the tie. In the analyses given by Mr. Noyes, on page 352, the percentage of chloride at the center is but little smaller than at the ends. Table No. 13 shows very much more in the edges than in the center; but this is probably due to the test

Mr. Curtis. having been made soon after injection. The statement is made that the chemicals diffuse themselves throughout the timber after the treatment has ceased, and while still in solution before the water has evaporated. No analyses are at hand, however, to prove this; but there is no doubt of the uniformity of the injection.

Generally, no measures are taken to protect ties where cut or bruised after treatment. Undoubtedly, it reduces the life when ties are thus injured, but, the treatment not being a superficial one, it is perhaps not as serious as with some other treatments. Naturally, the more surfaces open to attack, the more rapid the failure.

The author does not say that the transverse crushing is a negligible quantity; in fact, he expressed himself as of the contrary opinion.

It is to be regretted that Mr. Haines has been able to find so few facts in the paper; it is only necessary, however, to call attention to his misunderstanding of Mr. Kruttschnitt's experience. The treatment of ties on the Atlantic System was begun in 1886 under his direction, and he has given it his continued attention ever since; so his statements date from that year, not from 1894, as Mr. Haines states.

The data Mr. Boggs gives for the creosoted ties on the Houston and Texas Central Railroad are valuable and tend to confirm the general opinion of the value of creosoted ties, when properly treated. It is feared, however, that it is a hopeless task to endeavor to persuade our railroad officers to spend 40 or 50 cents for treatment, however good the results may be.

Apparently, the experience with Burnettized ties, cited by Mr. Boggs, has little bearing on the value of such treatment; the material used being so poor, or the treatment so defective, as to permit physical failure rather than decay.

The notes regarding Swiss practice, as given by Mr. Wagner, are interesting; but the American public is suspicious of foreign data unless they are so complete as to permit of an independent opinion being formed. The conditions may be entirely different or the conclusions reached may be based upon an insufficient number of facts; and unless the information comes from one who has investigated the situation personally, and who is known to our railroad officers, it is doubtful if any foreign data will get much of a hearing before them.

The near future will, no doubt, give us more men like Mr. Noyes, who can speak from long-continued close contact with the question. He emphasizes one of the difficulties mentioned in the original paper, viz., the necessity for sound material. Many ties treated in the past have been dead before they were treated, and should have gone into the fire instead of the cylinders. Careful inspection by experienced men will greatly decrease this risk, however; and the rule should be to reject improper material whenever found, regardless of the number of inspectors it may have already passed.

Mr. Curtis.

The reason given by Mr. Noyes for the necessity of seasoning after treatment by the zinc-tannin process, viz., to permit the particles of artificial leather to lose their water and become hard, seems rational. There is no doubt, in the author's mind, of the necessity for this seasoning with either of the zinc-chloride processes, but it would seem especially desirable with the double treatment.

The author has just completed an experiment to demonstrate what difference in the rate of absorption there might be in timber treated in various ways. A tie was selected, cut into four pieces, each weighing exactly 31½ lbs., and numbered 1, 2, 3 and 4. No. 1 was steamed 3 hours and placed in a vacuum of 20 ins. for ½ hour. No. 2 was treated the same, and, in addition, was under 100 lbs. pressure in zinc-chloride solution for 3 hours; in other words, it was Burnettized. No. 3 was treated like No. 2, and, in addition, received the glue and tannin treatment. No. 4 was left in its original state. All pieces were then placed over the boilers, where they were subjected to slight heat only. After 19 days they were weighed again, and then all were immersed in water. The resulting absorption, after 27 hours, was 3½ lbs. for No. 1, 3½ lbs. for No. 2, 2 lbs. for No. 3, and 3 lbs. for No. 4.

No conclusive judgment can be based on one experiment, but this seems to indicate that glue and tannin are effective in decreasing the absorptive power of timber. A tie treated with these specimens, and given the full zinc-tannin treatment, weighed 163.5 lbs. before and 195.5 lbs. after treatment; and two weeks later, after exposure on the ground, part of the time in the rain, weighed 174 lbs., showing how rapidly ties lose the water of injection.

Mr. Noyes refers to 305 experimental ties treated at Las Vegas and placed in the tracks at North Topeka in 1885. In 1891, 99.3% were still in service, and only 40 showed signs of decay. Mr. Noyes says that in 1895 all those on the 4°-curve (153 in number) were taken out, principally because of wear under the rail and spike-killing. Of the 152 on the tangent, only 22 were removed, leaving 130 still in. On October 31st, 1896, 81 were still in track. In a letter to the author, dated November 17th, 1898, Mr. Dun, Chief Engineer, stated that 81 were then in track, but that they were in bad condition, and would soon have to be removed. Mr. Noyes states, also, that the average life of white oak ties at this point is but 7 years. These treated ties were the New Mexico mountain pine, having a life, untreated, of from 4 to 5 years. The treated ones at North Topeka lasted 10 years on the curve, and the average life of those on the tangent is 12 years, thus—

22	ties removed in 1895, after 10 years	220	years.
49	" " " 1896, " 11 "	539	"
81	" still in track, 1898, but considered worn out, after 13 years	1 053	"
152	" at average life of 11.92 years	1 812	"

Mr. Curtis. Mr. Wallace expresses very well the view which railroad operators take of all questions connected with their work, and it is a view with which the author has no disposition to find fault. To begin the use of treated ties on a large scale means either the investment of a large sum of money in a plant, or the incurring of a large liability to private works for a term of years. When it can be demonstrated, however, that good results have been secured on at least two large roads, the element of uncertainty is very largely removed. Even if the experimental ties laid at La Junta and Topeka, described on page 306, and those at North Topeka, just referred to, are rejected as being only experimental, and those on the Ft. Wayne and the Duluth and Iron Range Railroads as being too recent, there remain the records of the New Mexico Division of the Atchison, Topeka and Santa Fé Railroad, which show that during 21 months ending December 31st, 1898, 27 694 of the ties treated in 1885 (being about one-fourth of the total number treated that year) were removed from the tracks, leaving a considerable number still in. The renewals on this division of 380 miles for the 5 years, 1893 to 1897, averaged but 217 per mile per year, while with the use of untreated oak ties, a renewal rate of 350 per year would not be excessive. Then on the Southern Pacific Railroad 93% of the ties treated 8 years before were still in service. The author has recently received from Mr. Kruttschnitt data bringing the records of this road down to July 1st, 1899. In Table No. 8, the percentages of ties July 1st, 1899, remaining in track were: After 11 years, 20%; 10 years, 78%; 9 years, 85%; 8 years, 94%; 7 years, 97%, etc. In Table No. 9, the average age of ties removed from January, 1896, to July, 1899, was 8 years 6 months, instead of 7 years 11 months. In Table No. 10, the average life of ties removed during year ending July 1st, 1899, was 8 years 6 months for the Glidden District, and 9 years 7 months for San Antonio District. This shows a continued increase in the average life of ties removed.

Such results cannot be called experimental.

If Mr. Churton will re-read the paper, he will find on pages 304 and 305 descriptions of the three processes in use in 1885; that the word "vat" occurs but once and then in a quotation; and that it is not admitted that the amount of chloride on the Southern Pacific was too great, but, in the author's judgment, was less than is desirable.

The comparison of the action of zinc on wood fiber with that of iron on cotton, he will have to leave to the chemists. An excess of iron in the zinc will most certainly injure the tie, and a very small amount of it constitutes an excess. Mr. Churton's idea as to what constitutes sound timber when cut down, differs from all authorities known to the author. The sap is a very essential element to a growing tree, as blood is to a living man; but the general impression is that a tie has the same use for sap that a mummy has for blood. If Mr. Churton knows of any treating works which have treated timber successfully without the

use of heat, it would be interesting to know of it. It would probably be an improvement if it were possible. Mr. Curtis.

Mr. Rowe's remarks about a surplusage of chemicals are deserving of consideration, particularly in view of his long experience in preservative work. It is well to remember, however, that there is no evidence that too much has been injected in recent years in this country, to the detriment of the timber; there is good evidence that it does gradually leach out; and the presumption is that the only undesirable result, if any, of a heavy injection, is the extra cost of the treatment.

It certainly seems to be the part of conservatism to inject up to ½ lb. of dry chloride per cubic foot of timber, when such an absorption can be obtained. The author takes the same position in regard to the zinc-tannin process, as compared with Burnettizing, proper. The additional cost of the glue and tannin is justified in view of the probable additional life of the ties. It will be time to abandon it when the Southern Pacific Railroad has demonstrated that it can get as long life from its Burnettized ties in Oregon and Northern California as others get from the zinc-tannin process in similar climates; or when some superior treatment is offered.

Replying to Mr. Eayrs, insufficient data was obtainable to cover bridge ties and timbers. So far as the author knows, no works, except those of the Atchison, Topeka and Santa Fé Railroad at Las Vegas, are treating such material with zinc chloride. Mr. Noyes writes that he is treating all their piling and bridge timbers, excepting the stringers, which are Oregon fir, and very durable in their natural condition; the piling being used both in wet and dry situations. The results are reported as satisfactory. There is an idea prevalent in the minds of some who are familiar with the treatment, that it is not adapted to timber which is exposed, as are bridge ties. It is supposed that the zinc may be more readily washed out under these conditions; but no evidence to support the theory is produced; and in the author's judgment it is based on a fallacy. So far as the moisture is concerned, he believes that less damage will be done to bridge ties than to track ties. He would not advise the use of Burnettized timber for stringers but believes it is well adapted for use in all other parts of viaducts or bridge floors. Its value for piles exposed to water in streams remains to be demonstrated.

Mr. Eayrs is in error as to the expense of dapping and boring ties. In connection with treatment, the ties can have the rail seat dressed and the spike holes bored for about 3 cents per tie. This would, undoubtedly, add materially to the life of the ties, and is something which will be done in future years, no doubt. The works at Las Vegas had a dapping machine in service some years ago, but its use was discontinued, apparently, because it was considered unwise to be too progressive.

Painting bridge ties will undoubtedly increase their life, provided the timber is thoroughly dry when painted. Otherwise it would seem

Mr. Curtis. inimical rather than beneficial. If the ties have been left too long unpainted it seems doubtful as to its advantage. If the injection of chemicals entirely throughout a stick is unable to prevent its destruction if decay has begun, what is the probable chance of doing so by the application of two or three coats of paint to its surface? It would be dangerous to draw any general conclusions as to the effect of paint on ties from the success with the long-leaf yellow pine at St. Louis. Poorer or different woods would probably show very much less satisfactory results.

Mr. Selby has worked out a very convenient and complete table of the relative costs of various ties. Where first-class oak ties can be purchased for only about 10 cents more than it is necessary to pay for a tie suitable for treatment, there is no inducement to adopt the latter, unless a very short life is being realized from the oaks, as on the Atchison, Topeka and Santa Fé Railroad, or where the climatic conditions are such as to justify the very cheap work of the Southern Pacific. The author depends more on the advancing prices of the oak and cedar ties than on economies demonstrated by means of calculations of compound-interest costs, to create a demand for treatment works; not because the latter are necessarily deceptive, but out of deference to the conservatism of officials.

The author desires to make but one criticism on Mr. Chanute's discussion, and that is, in reference to the use of tamarack. Mr. Chanute found it impossible to get as uniform absorption of the chemicals with this material as with hemlock. Of course, the object of treatment is not, primarily, to furnish a market for the zinc product of the country, but to increase the average life of all the ties treated. Before the use of treated tamarack ties is condemned, it is desirable to show that the average life secured is not satisfactory. This has not been done, and, at present, cannot be done; in fact, the evidence is rather to the contrary. Mr. Chanute being the owner and operator of private works, naturally desires to treat those timbers which will take the treatment readily, and does not feel called upon to risk his reputation to demonstrate that even a refractory material will give good average results. The author believes that tamarack can be treated satisfactorily and profitably.

Since this paper was presented to the Society three of our large railroads have started the construction of treating works. The Chicago, Burlington and Quincy Railroad has a two-cylinder plant under construction in the Black Hills of South Dakota; the Great Northern Railroad a three-cylinder plant, and the Mexican Central Railroad a three-cylinder plant in Mexico. Mr. A. A. Robinson, president of the Mexican Central Railroad, was the General Manager and Chief Engineer of the Atchison, Topeka and Santa Fé Railroad when the Las Vegas plant was built in 1885, and it was due to him that the road adopted the treatment. The construction of the proposed plant in Mexico is the best evidence of his satisfaction with the treatment.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

Note.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 862.

ELECTRICITY VERSUS STEAM FOR BRANCH RAILROAD LINES.

An Informal Discussion at the Annual Convention, June 30th, 1899.

Subject for Discussion.

"What are the economic conditions under which Electricity may be profitably substituted for Steam in the operation of Branch Railroad Lines, and what are the engineering requirements to be considered in such substitution?"

By Messrs. H. S. Haines, H. G. Prout, Oberlin Smith, W. B. Reed, G. S. Davison, W. L. Webb, J. James R. Croes, W. J. Baldwin, D. C. Jackson, Gustav Lindenthal, Edward Wegmann, Robert Moore, C. W. Buchholz, A. P. Davis and Charles H. Davis.

H. S. Haines, M. Am. Soc. C. E.—As the speaker is responsible for the submission of this subject for discussion he will undertake to state the purpose he had in view in suggesting it. Mr. Haines.

Electricity has been gradually introduced as a motive power on railroads, first as a substitute for horse-car lines in cities; then into continually widening areas, with the extension of such lines into suburban districts; all the time offering more formidable competition to steam railroads, until it has become a question as to its substitution for steam on some of these very lines.

This problem has been attacked by one or two of our principal railroad systems in what may still be considered an experimental way,

Mr. Haines. and it would be a matter of public utility if the results so far obtained should be gathered together and be made more accessible by publication in the *Transactions* of this Society.

The profitable substitution of electricity for steam in a railroad motor probably varies with its application, whether to the traffic on a strictly suburban branch, or to a branch in a rural or manufacturing district. There is also to be considered the advisability of its application alternately with steam on the same line. The conclusion seems to have been reached that this substitution can be more profitably made where it is desired to move single cars frequently rather than where it is required to move heavy trains at infrequent intervals.

The speaker thought of this subject because of a case that occurred in his own experience. In planning quite a long main line, he had occasion to provide for several branch lines to reach factory sites or manufacturing villages, at varying distances from the main line, where a mixed train making one or two round trips per day might meet all the requirements, at least for a year or two after the branch lines were opened. In one case it would be practicable to obtain current from a factory where it was generated by water power. In this way his mind was directed to the subject, but he was not sufficiently familiar with the economic conditions affecting the transmission of an electric current and its use as a motive power to determine the question satisfactorily to himself, and hence its reference to the Society for discussion.

The speaker is aware that there is a phase of the discussion of this subject which is more appropriate to a conference of expert electricians, but it is not that phase which he has presented here. It is distinctly as an engineering problem that he asks for a solution. There is an apparent disinclination on the part of civil engineers to consider, as appertaining to their domain, the application of electricity in connection with engineering works, in the same way that they accept and discuss questions involving the use of water or steam power.

As a help to himself, and to others similarly interested, the speaker would like to have a discussion of the conditions under which it would be advisable to operate by electricity a branch line to a steam railroad, whether in a suburban, a manufacturing, or a rural district; the conditions including the length of the branch, the character and frequency of the traffic, the generation of the electric current by water power or by steam, etc.

The subject should be considered, not only with reference to lines with a large traffic, but also as applied to branch lines operated only by one locomotive with a single train crew, making one or two round trips daily.

If these conditions can be determined with approximate accuracy, the field for the application of electricity will be accordingly extended,

and perhaps it will thereby become more feasible to provide traffic facilities for communities, to which they cannot be profitably extended with steam as a motive power.* Mr. Haines.

HENRY G. PROUT, M. Am. Soc. C. E.—This question is pre-eminently one of traffic conditions rather than of strictly engineering conditions; although engineering covers traffic conditions. Under such requirements as Mr. Haines has suggested, *i. e.*, branch lines with infrequent trains, it would not be advisable to try to substitute electricity for steam locomotives; and it is quite likely that during the next few years light steam motors for such work will be developed. Mr. Prout.

The costly installation of the New York, New Haven and Hartford Railroad at Hartford and Berlin, Conn., will probably be found to be a very expensive and unremunerative experiment. It is said that about half of the machinery in the power house is lying idle and that one-car trains are run at intervals of twenty or twenty-five minutes. Under such conditions it is impossible to pay the interest on the cost of the installation. In other words, if the traffic is in units of considerable size at infrequent intervals the work can be done more cheaply by steam locomotives than by electricity.

A locomotive which will develop say 1 000 H.-P. will cost (we will assume) $10 000, and a power house and plant to develop 1 000 H.-P., will cost perhaps $80 000. Thus it will be seen that the interest on $70 000 has to be earned, or accounted for in some way by the superior economies of the electric installation.

It may be said that there will be a great saving in the fuel account, but careful analysis, made within the last three years, has shown that the fuel will cost as much per horse-power delivered at the rim of the driving wheel for electricity as for a steam locomotive. Although in a fixed plant cheaper fuel can be used, and with all the appliances of automatic stoking, etc., can be burned more cheaply than in the steam locomotive, yet the losses in efficiency of the generator, in transmission, and in the motor, are sufficient to overcome that saving, and by the time the power is delivered at the rail it will have cost as much in coal as by the locomotive. This seems to be a safe general proposition, although Mr. Baker, General Superintendent of the Manhattan Elevated Railroad, has lately said that many improvements have been made within the last year or two, and greater efficiency is obtained from the generators and motors. But even if a saving in coal of, say, 50% can be made, yet, after all, the fuel item in the operation of a railroad is comparatively small. Assuming the fuel element to be about 13% of the total operating expenses, then, even if the saving in this item is 50%, it is evident that little has been saved toward paying the interest on the expensive electric installation.

If the traffic is such that its efficiency, or commercial development,

* For further remarks by Mr. Haines, see p. 386.

Mr. Prout. requires a minute subdivision of the units, that is, a division into very small units running very frequently, in order to induce traffic to move, then, of course, electricity will win, not because it is cheaper than steam, but because it is adapted to such minute subdivision.

Mr. Smith. OBERLIN SMITH, M. Am. Soc. C. E.—For several years past the speaker has publicly advocated the use of small electric motors for driving machine tools and other independent machines. Although the idea at first appeared somewhat chimerical, individual motors are now extensively so used, the system having mostly passed beyond the experimental and transitional stages.

Railroad motive power is now in a state of transition. Within the last ten years the ordinary street trolley line has grown from a mere experiment into a great industry, which has not only performed the work of the horse-cars it displaced, but has created an enormous traveling habit. The time for the universal use of these motors on railroads has not yet arrived, but the speaker believes that it will come eventually, in consequence of discoveries which will cheapen the use of fuel and simplify electrical machinery; and also because such a method will result in greater convenience to travelers, by reason of more smoothness of motion, freedom from dust, smoke, cinders, etc., and facility for running single cars (or short trains) at much more frequent intervals than now. It therefore follows that the use of electricity on roads which are at present run by steam, will largely increase travel—and the habit of traveling. Such improvements make car riding a pleasure rather than the duty that it now often is.

The foregoing remarks apply to travel on surface roads, at speeds much the same as now practiced. The change in method has already begun, and its adoption by all railroads is simply a matter of relative cost—in construction, maintenance and operation.

It would seem probable, however, that in the next generation, if not in this, an entirely new type of high-speed railroad will be developed. Mechanically, the world is ready to begin it—financially, not yet, for its cost would be great. Upon some special type of track, confining the cars absolutely against derailment, a speed of 120 to 150 miles per hour, and possibly more, can doubtless be obtained; and it would not seem to be a very wonderful feat to cover the 90 miles between New York and Philadelphia in half or three-quarters of an hour. There are no physical or mechanical obstacles which cannot be overcome with our present knowledge and some experimenting with details.

The danger of collision with other objects can be avoided by the use of an elevated roadway with absolutely no crossings, turn-tables or switches; and with the trains run upon a positive electric block system, in one direction on one track.

The questions of derailment and collision being obliterated, the

speed of trains will be limited only by air resistance and by the strength of the wheels to resist centrifugal force—if wheels are used. If the cars are made long, narrow, pointed, very smooth and properly jointed together when in trains of more than one, they can probably be run at high speed without encountering such great air resistance as would make the cost of power too great. Such cost, however, must not be based upon present train weights in relation to live load. Mr. Smith.

In an article in the *Forum*, for January, 1891, the speaker showed that the dead weight carried on American railroads, in many cases, amounted to 6 tons per passenger on trains of Pullman cars, and that a load of 1 or 2 tons per passenger is very common. From a mechanical standpoint this excessive weight is not necessary. The bicycle has shown that a person can be carried over a smooth dirt road with 20 lbs. of dead load, and on an iron pathway this can be further reduced; if riding on a tricycle covered from the weather, as in a car, this load might be 150 lbs., or about his own weight.

In this respect the bicycle has been an object lesson, and has shown that it is possible for a vehicle to be so delicately made, to have so much refinement in its mechanical design and composition, with ball-bearings, steel suspension-spokes, tubular frames, etc., that our old notions of wagons and cars appear absurd in comparison, especially when we carry 6 tons, or even 1 ton, of dead weight for each passenger.

It has been demonstrated that a heavy steam locomotive and train can travel upwards of 100 miles an hour, with all the disadvantages of a tremendous load not on the drivers, and a degree of uncertainty as to whether or not the train will stay on the rails. If, however, such a train could be made safe upon a surface road, at the speeds under consideration, and did it not cost too much for power, yet there are limitations of speed in respect to getting steam through the ports quickly enough and in working reciprocating members—as pistons, connecting-rods, etc. These limitations do not exist with rotating electric motors.

It would seem likely, therefore, that some of us may live to see some such railroading as has been indicated—a service safe, rapid and frequent, developed at first over short distances between very large cities. Should we succeed in adapting to the axles some form of suitable roller-bearings, we will save as much power as possible under the circumstances. Should we, further, be fortunate enough to find it practicable to equip our wheels with rubber tires, or some yet to be discovered equivalent, we then might get a smoothness and quietness of running which would indeed proclaim the advent of the ideal railroad.

One reason why electricity works better upon steep grades is that, with the best modern systems, every wheel in the train can be made a driving wheel. A train can be comparatively light, without the dead

Mr. Smith. weight of a locomotive, and yet have sufficient traction. With a steam train we not only have the cars very heavy, especially Pullman cars, but we have the big locomotive itself to drag up grades, with only the adhesion due to the weight of part of the locomotive, such as happens to be on the drivers. This is evidently very different from being able to have a very light train and making every wheel a driver, with every pound of the train and load contributing its share of adhesion.

Mr. Reed. W. B. Reed, Assoc. M. Am. Soc. C. E.—The discussion, up to this point, has referred mainly to passengers. The question of carrying freight in combination with passengers, upon electric lines, should also be discussed. Generally speaking, it has been found that for short distances passengers can be carried more comfortably and more cheaply by electricity than by steam, but in the carrying of freight little has been done by electricity as yet. That is a feature which will grow in the future.

When freight is to be carried over passenger lines by electricity it will have to be taken through city streets, or at least over highways, and the rails should be designed to meet the conditions of electric cars which have shallow flanges; freight cars with deeper flanges, and which vary more or less in gauge, requiring not only a deep groove but a wide flangeway; and the ordinary vehicle traffic.

Mr. Davison. G. S. Davison, M. Am. Soc. C. E.—About 8 or 9 years ago the speaker was in charge of the construction of a small branch line near Pittsburg which was required to connect a small manufacturing town with the Pittsburg and Lake Erie Railroad and the Pennsylvania lines. It was an independent corporation, controlled by manufacturers in the town, and it was equipped at first with steam power. The president of the company conceived the idea that with such cheap fuel as is to be had in that neighborhood, and in view of a proposed change in the future from steam to water power, it would be proper to operate this branch by electricity instead of steam. He experimented slowly, and equipped the line as a trolley road, but separated the passenger and freight traffic, and carried the former by electricity and the latter by steam locomotives.

The speaker does not know the results of the operations of this company, but judging from later occurrences he believes that the separation of the passenger and freight service was found to be a mistake. In this case the company had ideal conditions for cheap fuel, and had also its own electric power plant. It was not necessary to install a new plant, but simply to add to the old.

The road was afterward purchased by the Pittsburg and Lake Erie Railroad Company, and is now being operated as a branch of that road. The company, on acquiring this branch line studied its operation to determine whether it should be conducted as an electric line in part or as a whole, or whether it should be operated as a

steam road. As the electrical equipment was discarded and as the road is now being operated by steam, it may be assumed that electricity was found to be uneconomical. The causes which led to this change are not known to the speaker, but he suggests that possibly the trolley equipment might have been found very inconvenient when operating the road by steam as a freight line. The third-rail system may prove to be more practicable than the trolley in such cases, but the latter must be excluded on account of its inconvenience. Mr. Davison.

In the vicinity of Pittsburg there is a large mine which is operated for some 7 miles back from the point where the coal is brought to the tipple; the means of bringing the coal to the opening being virtually a small branch railroad. This branch passes through several tunnels and has been operated as a steam railroad for a number of years. At the present time the first 5 miles of the road is being operated as an electric line, using the trolley system, and on the remainder of the line, next to where the coal is brought out to the tipple, steam locomotives are used. The owner stated that the operation had been planned in that way, and that they had been justified in hauling the coal the first distance by electricity, because the use of steam locomotives in the mines was detrimental to the health of the men. But, taking full account of the cost of installation, power, etc., they had not found that it would be cheaper or better to continue the use of the trolley to the tipple.

The fuel of this power station is delivered directly from the mine into the boiler house, and yet, on the 5 miles operated by electricity, the saving on fuel is so small that the expense of an electric equipment for the remaining 2 miles would not be justified.

As the topic for discussion refers only to the operation of branch roads, in connection with a main line, Mr. Smith's proposition to attach a motor to every axle on the train would seem to be impracticable as it would involve the equipment of every car of the trunk line so that it could be used on the branch.

Walter L. Webb, Assoc. M. Am. Soc. C. E.—The relation of grades to the question of steam or electric operation has not yet been touched upon in this discussion. On a steam railroad, a grade of 2% is considered steep, while 4% marks almost the limit of working on such roads; yet on many of the electric railroads in the United States there are grades of 10% and even steeper. On steam roads which are steep in some places and flat in others there is a great waste of power. This occurs, not on the heavy grades where the engines are hard worked, but on the light grades where the great reserve power—at those points doing practically no work—has to be maintained for service on the heavy grades. On an electric road the amount of power used is almost proportional to the work to be done, and this is an important element in its economy. On a railroad in a mountainous region it Mr. Webb.

Mr. Webb. should be possible to adopt a cheaper form of construction than is necessary for a steam road with its requirements of low uniform grade, and to use freely very steep grades where desirable. Assuming a fair amount of traffic, so distributed as to keep the road in uniform operation, electricity will, in many cases, prove more desirable than steam locomotives, taking into consideration the interest on the cost of the construction and installation, and the expense of operation, and this will be true for these reasons alone, to say nothing of the possible economies consequent on a utilization of water-power.

Mr. Croes. J. James R. Croes, M. Am. Soc. C. E.—The economical generation of power, either steam or electric, implies a constant and continuous operation of the generating plant. When variable power is needed, the generating plant must be constantly operated at its maximum efficiency. When the work to be accomplished can be distributed over a continuous period of 10 to 24 hours, the power can be more economically generated by an indirect application through a stationary plant than by a direct application of the fuel to the motor, as in the steam locomotive. Where, however, the work involves a spasmodic application, occupying only a limited amount of time, at irregular periods during the day, the generation of power by an independent motor, capable of producing in itself the power needed, is more convenient and more economical than its production by a central plant.

Steam power at the present day can be generated spasmodically more efficiently than electric power. The requirements of railroad service on a line of limited traffic are spasmodic, reaching a maximum at certain hours; and until a storage battery has been devised in which the power produced continuously can be stored and used for spasmodic application, which application shall satisfy a maximum call greater than the average efficiency required of the generating plant, electricity cannot be applied more economically than steam. At present, it seems to the speaker that where the work is spasmodic, steam power is, and must be, more economical than electric power, for branch railroads.

Mr. Baldwin. William J. Baldwin, M. Am. Soc. C. E.—In many States the warming of cars is obligatory by law, and the cost of heating should therefore be taken into consideration in a comparison of railroad operation by steam and by electricity. About three years ago the speaker had occasion to test an electric heater. There was no unit of measurement to be determined, but the heater furnished was found, by comparison, to agree with 28 ft. of steam-heating surface. It was found that 28 ft. of steam-heating surface will condense about 7 lbs. of water in an hour, or about 0.25 H.-P. The electric heater, practically the equivalent of the heating surface mentioned, required between 6 and 7 electric H.-P. The result of this test indicates that warming by electric heaters requires 27 or 28 times the horse-power needed to warm by steam.

D. C. Jackson, M. Am. Soc. C. E.—There is no mystery about this matter, and Mr. Croes has well stated the case. It is more economical —not more efficient—to generate power for operating a railroad or a group of manufacturing establishments at a central station only when the power can be used continuously over long hours, for the reason that when the traffic on the railroad or the load on the motors at the manufacturing establishments is continuous, the demand on the central station for power is continuous, and the cost for interest and labor per horse-power hour developed by the station is brought to a reasonable figure. Where traffic requirements are intermittent, it is more economical to operate a railroad by steam locomotives. If a plant of 1 000 H.-P., installed to operate a branch railroad with freight traffic, is called on for its output for only 3 hours of the day, 24 hours of interest charges lie against the 3 hours of output. There are at the station, daily, possibly 10 hours of full labor and 14 hours of one-half or one-third labor, and if this is charged against the 3-hour output, it makes the cost of operating the station, per horse-power hour, enormous. Whether or not, electric power may be economical in various cases is, as Mr. Prout has said, a matter of the distribution of traffic. Mr. Jackson.

These remarks apply equally to electric power plants where the prime power is derived from steam engines or water wheels. The larger cost of development of a water-power plant will often cause an excess interest charge which is equal to the cost of fuel in a steam plant designed to do the same work, and the average small water-power plant does not embrace such economical advantages as are generally ascribed to it.

The case of a main-line feeder, as put by Mr. Haines, is a special case under the general proposition, and under some circumstances it is possible that electric power can handle the traffic most economically. These conditions would assume a number of main-line feeders, located reasonably near a given point, so that the entire load of all these branches may be carried by a central station, using a fairly high transmitting pressure. The traffic should be broken into units somewhat smaller than would be operated ordinarily by steam power direct, and schedules of train movements should be so arranged that the station may be fairly well loaded during at least 12 hours. If the schedules are so arranged that once or twice during the day a single mixed train will be running simultaneously over each of these branches, the speaker doubts if electric power, from the point of view of dollars and cents, can ever compete with the steam locomotive.

Referring to the requirements for the electric heating of cars, some of the western electric street railways make provision for turning into the heaters during cold weather, as much power as is required to move the cars. In other words, in very cold weather, a station output of double that necessary to handle the cars is anticipated.

Mr. Lindenthal. GUSTAV LINDENTHAL, M. Am. Soc. C. E.—In discussing this question an important feature is the ability to store electric power. In recent years several electric power plants have been so constructed that, when running at a uniform rate, the power generated in excess of current demand could be stored up in secondary batteries acting as reservoirs, to be given out again whenever the demand for power exceeds the capacity of the plant.

A railroad train in descending a grade wastes power in grinding up brake shoes, and heretofore this power has not been stored in any form. It is possible to generate and store that power in the form of electricity, which can be used again in hauling trains or cars up an ascending grade, or for lighting or heating purposes. The speaker believes that the feature of storing electric power should have an important bearing in any comparison of the economies of locomotive and electric traction, particularly in mountainous countries where coal may be dear and water-power cheap.

Mr. Wegmann. EDWARD WEGMANN, M. Am. Soc. C. E.—The speaker objects to the statement that steeper grades can be used on electric tramways than on ordinary steam railroads. While it is undoubtedly true that steeper grades are used on trolley lines than on ordinary railroads, this difference should not be credited to electricity as against steam, but to the placing of a motor under each car. If a steam engine were applied to a car in this manner, the car could be driven up any of the grades to be found on trolley lines.

Before any trolley lines had been constructed the speaker had spent two or three years in assisting an inventor in developing and introducing a steam street car, which was run successfully in Brooklyn. This car, which was propelled by a small three-cylinder steam engine, placed between the axles and suspended from the car frame, was able to run up 10% grades.

While such a car might be used to advantage, where only a few trips had to be made each day, it had several disadvantages compared with trolley cars: 1st, The weight of the boiler was objectionable; 2d, A licensed engineer was required for each car. One of the great advantages of the trolley system is that men of ordinary intelligence can soon be taught to run the cars.

In the case of pulling a train by a special motor, the grade that could be overcome depended, as was well known, on the weight on the driving wheels, the rolling friction of the wheels, and the weight of the train. It made, therefore, no difference as regards the grades, whether steam, electricity, compressed air, or any other motive power were used to propel the motor.

Mr. Moore. ROBERT MOORE, M. Am. Soc. C. E.—Each particular case of this kind must be considered by itself. In the case indicated by Mr. Haines, in which the power is to be rented and paid for, presumably,

by a meter, the conditions of economy would be very different from those in which the railroad company would be required to install the plant, and bear the cost of the current whether it were being used or not. Therefore a proposition which would cover all cases might be very delusive. So far as a general proposition can now be made it must be substantially as indicated by Mr. Prout. Mr. Moore.

When the conditions are those of an urban traffic—a very large number of small trains—electricity has abundantly demonstrated its superiority over every other form of motive power; but when the conditions are very different from those of urban traffic—when the trains are infrequent or heavy—the speaker doubts its economy, and sees no indications that it is likely to be economical in the near future.

Regarding the superior economy claimed for the electric motor on heavy grades, the point cited by Mr. Wegmann is a sufficient answer. The power is developed only by the traction of the driver on the rail, and that is a function of its weight; thus, there can be no saving in that respect. But there is another point which should not be lightly regarded, the great danger, in operating on very steep grades, of the failure of brake power. Any such failure means disaster, as many who have had occasion to operate heavy grades well know.

In fact, if, with a possibility of doing otherwise, any one is tempted to use heavy grades, the best advice to follow is that of *Punch* to those about to be married—"Don't."

C. W. Buchholz, M. Am. Soc. C. E.—The facts are against the assertion that the grades on an electric road can be steeper than on a steam road. The power of a locomotive depends upon its adhesion to the rail, and it makes no difference whether the power is steam or electricity. Mr. Buchholz.

A. P. Davis, Assoc. M. Am. Soc. C. E.—This question is, without doubt, one of the kind of traffic, and to one more familiar with hydraulics than with railroad work, it appears to be a matter of saving, not 20, 40 or 50% of the fuel, but all of it. It has been pointed out that electric power makes its best showing in the mining regions, and one of its chief advantages seems to be that on steep grades it requires less dead-weight than steam power, an important point in regions where steep grades are unavoidable. In mountainous regions the railroads, both main and branch lines, follow the streams, sometimes of heavy fall, where water power can be developed cheaply. In many cases in the West a water-power plant would not be chargeable with the total expense of the development, for whatever of diversion works and storage facilities are required are equally available as parts of an irrigation system which will receive the water after it has been used for power. Mr. Davis.

Two lines of railroad extend up the line of the Arkansas River,

Mr. Davis cross the Rocky Mountains and follow the Grand River down on the other side. There, where coal is much more expensive than in the East, is an immense amount of power, constantly going to waste, which might be utilized by the electric system to move those frequent and heavy trains. Undoubtedly, the time is approaching when a great saving can be made in equipping with electric power, not only branch lines, but main roads as well, through mountainous regions, where water power is available. Improvements in the transmission of power are still possible, and the utilization of water-power in railroad operation is mainly a question of transmission. As transmission methods are improved, and as railroad traffic in such regions increases, the advantage of electric over steam power will be demonstrated with a definiteness to which the probable advance in the price of fuel will add its contribution.

Mr. Haines. H. S. Haines, M. Am. Soc. C. E.—This question was suggested by the speaker because he considered it to be of practical moment, and because he desired to obtain from members of the Society an authentic expression of opinion as to how far, in the present state of the art, it was advisable to substitute electricity for steam as a motive power on branch lines of steam railroads.

If electricity is used on branch lines it must be used in connection with the vehicles in which passengers and freight are transported on the main lines. It is not only desirable to ascertain what can be done with electricity under this aspect of the case, but also what cannot be done. There is one general proposition that all will accept, viz., that in the present state of the art there is no doubt that electricity can be economically substituted for steam on a branch of a steam railroad where there are single units of transportation moving at frequent intervals, because the power can be used uniformly and continuously and because a large plant is not lying idle for the greater part of the time.

To state the cases in which electric transmission cannot be applied, a process of exclusion must be used; therefore, let it be said that it can be applied economically on branch lines for suburban purposes. If that is the case, it strengthens the steam railroads which have suburban branches competing with trolley lines. This seems to have been the idea which has prevailed in the minds of the management of some of the principal railroad systems operating out of large centers of population. The speaker recently had occasion to talk this matter over with some of the principal officials of the New York, New Haven and Hartford Railroad, who have probably had more experience in this matter than anyone else; and the impression received was that they are quite well satisfied with what they have done thus far in an experimental way, and they are extending the system. There is one point, however, which they have not yet determined, and that is, the con-

ditions under which the third rail should take the place of the trolley wire. Mr. Haines.

Admitting that electricity can be used economically on suburban lines the next question is: Can it be used on branch lines under other conditions? Judging from the preceding discussion the speaker believes that electric motors might be used to great advantage on branch lines in a manufacturing district, even for handling freight, if power can be obtained along those lines from manufacturers who have it to spare.

On a line running into a rural district it would seem hardly practicable to use electricity, unless, as Mr. A. P. Davis remarked, water-power could be obtained. That was one of the conditions in the case which was being considered by the speaker. The main line was for some miles parallel with a stream which it was intended to develop for manufacturing purposes; but the line ran along a ridge and the water-power was down in the valley. If branch lines could be run down into the valley the water-power there could probably be used more economically than steam.

The speaker need not discuss the question of heavy grades, as that has been covered so well by Mr. Buchholz. Unless by the use of electricity the traction can be made stronger than the mere adhesion due to the weight of the locomotive, that question need not be considered.

To sum up, it may be said that in the present state of the art it is only admissible to apply electricity on what may be called suburban passenger lines; except that it may be applied where power, for certain local reasons, may be obtained more cheaply than by means of the steam locomotive.

Charles Henry Davis, M. Am. Soc. C. E. (by letter).—While the subject for discussion seems to be confined to the application of electric motive power to branch steam railroad lines, the members speaking on this subject, at the Annual Convention, June 30th, 1899, are evidently interested in the problem as a whole and have not confined themselves to "branch lines." Claiming the same privilege, the writer transmits the following as possibly throwing some light on the future development in the use of electric traction, not only on existing steam railroad lines, but for new high-speed interurban transportation systems. Mr. Davis.

Reversing the usual order, some of the writer's conclusions are given prior to the discussion, in order that each step may be followed more readily. The figures and tables may be found to have some errors (of which the author will be glad to be advised), and, in many cases, are only approximate; so that they must not necessarily be treated as accurate in their details, but they are safe as to their conclusions, and can be relied upon in this respect.

Mr. Davis.

Conclusions.

(*1*) Steam railroads will, in the near future, handle their suburban and short-distance interurban passenger traffic and mail, express, baggage and light local freight carried in said suburban and interurban passenger trains, by electric motive power; and this, irrespective of whether operating expenses are affected favorably or unfavorably.

(*2*) Steam railroads will not, in the near future, handle their freight traffic (other than mail, express, baggage and light local freight carried in suburban and interurban passenger trains), and long-distance passenger traffic by any other motive power than steam locomotives.

(*3*) Steam railroads may, under exceptional conditions of large volume and great density of passenger traffic over distances longer than under (*1*) and shorter than under (*2*), handle it by electric motive power, but such cases will be infrequent.

(*4*) New railway lines, connecting very large centers of population, where frequent service at much higher speeds than can be attained now by steam locomotives on existing lines are conditions of success, will be operated by electric motors.

There are three conditions under which suburban and short-distance interurban traffic will be handled profitably by steam railroads converting to electric traction:

1. (*a*) Where units can be light and frequent, and operated over comparatively short distances.

(*b*) Where gross receipts can be so increased by the change of system and mode of operation as to pay for the increased investment and possible increase in operating expenses.

(*c*) Where competition of parallel electric roads compels the change, to save what traffic there is, irrespective of how operating expenses are affected.

In the future development of steam railroad systems they will eventually be operated jointly with surface electric railways, either through actual mutual ownership or by traffic contracts, leases, etc.

The above conclusions are obviously dependent upon what Mr. Prout properly defines as "traffic conditions," and not primarily engineering details or operating expenses.

Laws of Passenger Traffic.

The laws of passenger movement are not well defined, and many of them are illusive and hard to determine; the causes of loss or gain are often largely a matter of individual judgment, so that the following outline of them must not be taken as in any way exact.

People travel from one place to another from (1), necessity, and (2), pleasure or whim.

They are induced to travel more or less often according to:

1. Total Cost from Point of Departure to Objective Point and Return to

Mr. Davis.

EXPLANATION

Curve		Multiplier for Vertical Scale
1.	Total Miles of Track	1 000
2.	Annual Increase of Mileage	100
3.	Total Population of the United States	1 000 000
4.	Percentage of Increase of Population of United States	1
5.	Density of Population of the United States	1
6.	" " " " Massachusetts	2
7.	Total Urban Population of the United States	100 000
8.	Percentage of Urban to Total Population of the United States	½
9.	Total Population of New York, New Jersey and Pennsylvania	100 000
10.	" " " Massachusetts	100 000
11.	" " " New York City	10 000
12.	" " " Brooklyn	10 000
13.	" " " Philadelphia	10 000
14.	" " Served by N.Y.& P.R.R.R.	100 000

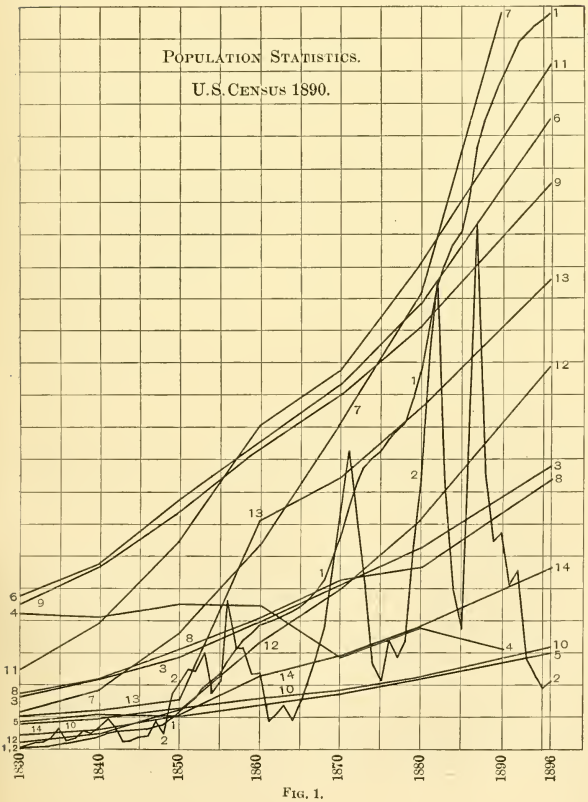

FIG. 1.

Mr. Davis. *Place of Beginning.*—As the total cost is reduced, travel, due to both fundamental causes, is increased; presumably less rapidly than the total cost falls. Note that the important factor to the passenger is total cost, and not cost per mile traveled.

2. *Total Time Consumed in Making the Round Trip.*—As the total time consumed is reduced, travel, due to both fundamental causes, is increased, presumably less rapidly than the total time falls. Velocity of transportation is not primarily effective in inducing travel, for it makes no difference to the passenger whether he be carried 80 miles in 30 minutes or only 12 miles.

3. *Total Conveniences Afforded the Passenger.*—These may be divided into:

(*a*) Proximity of departure and arrival points to possible passengers. As a "leave-at-your-door" service is approached, passenger traffic increases, but according to no known ratio to distance. Wellington laid down an approximate rule of loss of natural revenue for steam railroads of 10% per mile of removal from center of population as a minimum, 25% per mile as an ordinary maximum and a much larger percentage of loss, or even total loss, under certain conditions. Electric street railways have profited more by this kind of service, which they offer the public, than from any other reason; in furnishing it they give frequent and quick service, both of which are of the greatest importance in their effect on passenger traffic. Much less than a mile, however, will make or ruin the passenger traffic of a street railway.

(*b*) Frequency of the service. As the number of trips increases, so will the passengers, but less rapidly than the headway is shortened. A frequent service means less "total time" consumed.

(*c*) Character of terminals, stations, roadbed, equipment, and, in fact, all physical characteristics. That transportation system which offers, at the same rate and time, better physical conditions, which give comfort or even luxury to the passenger, will not only secure competitive traffic, but induce that which would not otherwise exist.

4. *Total Population.*—As the population served increases, the passenger trips per capita per annum increase, and somewhat faster than the inhabitants, unless modified by density and distribution.

5. *Density.*—As the density increases, it is probable that the rides per capita per annum also increase, but whether more or less rapidly is uncertain.*

6. *Distribution.*—A long, narrow town will give more rides per capita per annum than a square town having the same population.

* In an earlier publication the writer expressed the opinion that the increase was less rapid than the density, but wishes to leave this open for further investigation.

Mr. Davis.

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Total Miles of Track	10
2.	Annual Increase of Mileage	10
3.	Total Capital Stock, in Dollars	500 000
4.	" Bonded Debt " "	500 000
5.	" Capitalization " "	500 000
6.	" " per Mile of Track, in Dollars	500

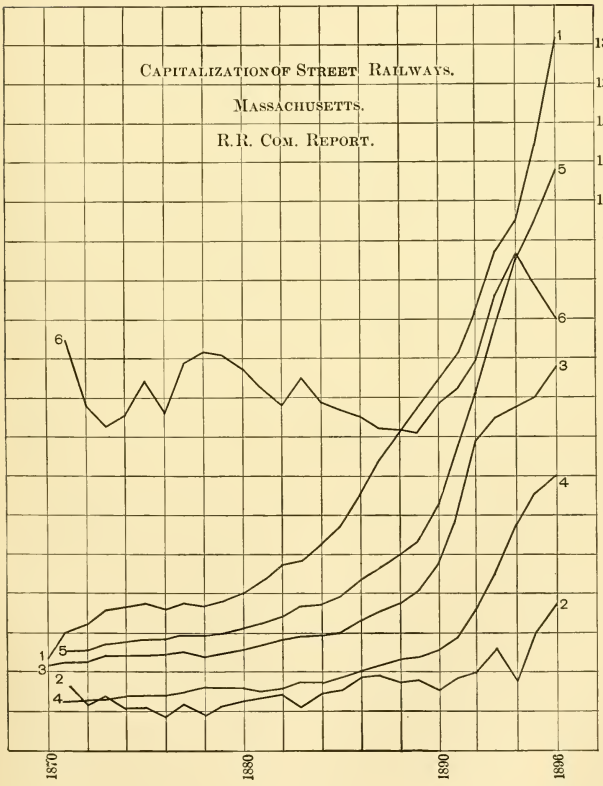

FIG. 2.

Mr. Davis. 7. *Character of the Industries and Population.*—The effect of various industries and the kind of population must have a decided effect on the passenger traffic, but probably according to no fixed laws, and certainly according to no known laws.

No "detailed defence" is offered for these laws other than what support they may obtain from the following discussion.

Any management of a steam railroad, in considering the question of adopting electric traction, will naturally turn first to data on operating expenses, for, if its use should lower the cost per train or car-mile enough to more than pay interest on the investment, then it would be of advantage, even though there were no other inducements, such as increased traffic. While comparisons might be made between the operating expenses of steam railroads, which are now well known, and such electrically operated roads as the Chicago Elevated Railways, or the third-rail lines of the New York, New Haven and Hartford Railroad, yet there are disadvantages in doing so which outweigh the apparent one of more similar conditions than exist between the ordinary street railway and the steam railroad. If it should appear that the operating expenses of electric street railways per car-mile are less than the cost per car-mile operated on steam railroads, one can be certain that electrical operation, applied to the latter, will result in reduced expenses. Whether or not this reduction will be enough to pay for the change is another question. For these reasons, a comparison between the street railways and steam railroads of the New England States has been taken as a basis in this discussion.

To exhibit the relative merits of the two systems of installations, the following divisions are arranged in tabular form and are treated in order. The prefixion of the word "electric" or "steam" indicates that the item belongs exclusively to one or the other system and has no counterpart in the other.

In comparing the steam and electric roads of New England, we are more likely to arrive at correct conclusions, as the population is more dense, the miles of road greater per square mile, there are many more examples to study, and the statistics are more accurate and reliable, and cover longer periods. The word "Railroad" is used to designate a "steam railroad," while "Railway" always indicates an "electric street railway," following Massachusetts precedent; the difference being that the former operates on its own right-of-way, while the latter occupies the public highway. This distinction is a desirable one to cultivate, and should be extended and used irrespective of the mode of operation; thus, a road operating by steam, electricity or any other motive power on its own right-of-way is a "railroad," and one operating on a public highway is a "railway."

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Total Miles of Track	100
2.	Annual Increase of Mileage	10
3.	Total Capital Stock, in Dollars	10 000 000
4.	" Bonded Debt " "	1 000 000
5.	" Capitalization " "	10 000 000
6.	" " per Mile of Track, in Dollars	1 000

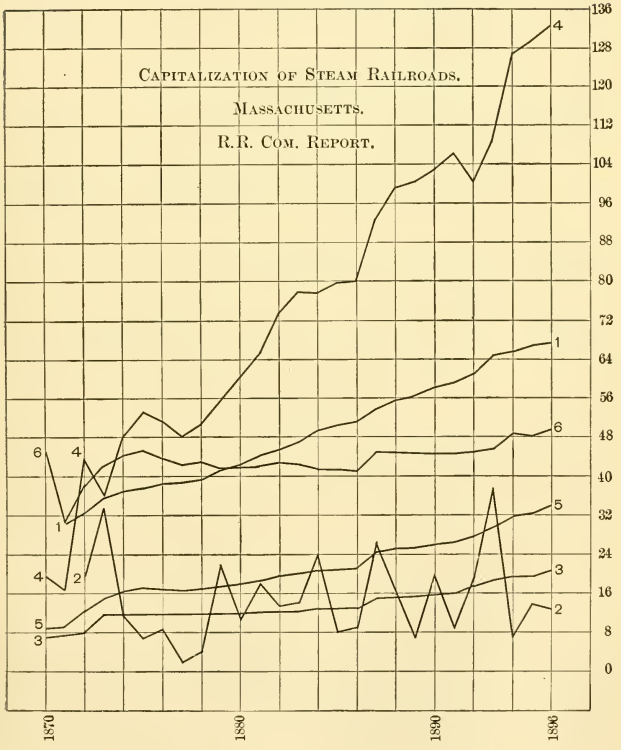

FIG. 3.

Mr. Davis.

TABLE No. 1.

1. FIRST COST.

(*a*) Right-of-Way and Real Estate.

(*b*) Construction.

1. Engineering and surveying.
2. Clearing.
3. Grading, ditching, etc.
4. Tunnels.
5. Masonry, culverts, bridges, trestles.
6. Fencing, cattle-guards, road-crossings.
7. Ballasting.
8. Ties.
9. Rails.
10. Track-laying, lining and surfacing.

(Electric) 11. Rail bonding.

(Electric) 12. Overhead-trolley line, or third rail.

13. Signals.
14. Telegraph.
15. Stations and all buildings (including buildings for central power stations of electric roads).
16. Terminals.

(*c*) Equipment.

(Steam.) 1. Locomotives and tenders.

(Electric) 2. Motor cars or electric locomotives.

3. Passenger cars.
4. Freight cars.

(Electric) 5. Electric feeder lines and ground-return circuit.

(Electric) 6. Central power stations (not including buildings which are considered under "Construction").

(*d*) General.

1. Discount on bonds.
2. Interest on bonds, to opening of road.
3. Taxes, to opening of road.
4. Office expenses, salaries, etc., to opening of road.
5. Contingent and miscellaneous, not itemized, to opening of road.

2. TOTAL EXPENSES.

(*a*) Maintenance and Renewal of Way and Works.

1. Repairs of earthwork to subgrade.
2. Repairs of track.
3. Repairs of fences, cattleguards, crossings, etc.

(Electric) 4. Repairs of ground-return circuit.

(Electric) 5. Repairs of overhead-trolley line, or third rail.

(Electric) 6. Repairs of electric feeder lines.

7. Renewals of rails.
8. Renewals of ties.
9. Renewals of ballast.

(Electric) 10. Renewals of poles, or insulation of third rail.

(Electric) 11. Renewals of trolley, or third rail.
(Electric) 12. Renewals of feeders.
13. Repairs of masonry, bridges, trestles.
14. Repairs of buildings.
15. Repairs of signals.

(*b*) Train Expenses.

(Steam) 1. Fuel for locomotives.
(Electric) 2. Fuel for power stations.
3. Water supply.
4. Oil and waste.
(Steam) 5. Repairs and maintenance of locomotives and tenders.
(Electric) 6. Repairs and maintenance of motor cars or electric locomotives.
7. Repairs and maintenance of passenger cars.
8. Repairs and maintenance of freight cars.
9. Use of foreign passenger and freight cars.
(Steam) 10. Locomotive services (wages).
(Electric) 11. Motor car or electric locomotive services (wages).
12. Passenger-train services (wages).
13. Freight-train services (wages).
(Electric) 14. Repairs and maintenance of power stations.
(Electric) 15. Power-station services (wages).

(*c*) Station, Terminal, Taxes and General Expenses.

1. Agents and station services (wages).
2. Station supplies.
3. Telegraph.
4. Taxes.
5. General officers and clerks.
6. Legal.
7. Insurance.
8. Stationery and printing.
9. Agencies and advertising.
10. Contingent and miscellaneous, not included above.

(*d*) Accidents, Loss and Damages.

1. To freight.
2. To passengers.
3. To property owned.
4. To property not owned.

(*e*) Interest Account.

(*f*) Fixed Charges.

1. Interest on bonds.
2. Rentals.

(*g*) Dividends.

3. GROSS RECEIPTS.

1. Passenger receipts.
2. Freight receipts.
3. Miscellaneous receipts.

1. First Cost.

Mr. Davis. We shall consider a new line under steam-railroad conditions, first, and then a change of system on an existing railroad.

The first cost of (*a*) "Right-of-Way and Real Estate" cannot be affected by any particular motive power unless the use of electric traction permits the use of sharper curves and heavier grades, thus avoiding large cuts or fills. While a coupled locomotive undoubtedly increases "curve resistance," this is not sufficient to be of any great moment. If electric motors are applied to every car axle, then trains of the same weight, moved by electricity, can surmount heavier grades than with locomotives or can pass over the same grades at higher speeds. Hill climbing is a matter of traction between driven wheels and rails, and the sustained horse-power of the motor or locomotive. With the power applied to every axle, there is no part of the load which remains unutilized for traction, but this cannot be done by the application of steam, and has, as yet, been only partially successful with electric motors. With the latter, their maximum power can be called upon for an indefinite time, because they do not give out at the top of a long, heavy grade. This advantage of electric traction is, however, only apparent; first, because long trains are not yet satisfactorily controlled where motors are put on every axle, and short trains of two to five cars have only half the axles supplied with power in present practice; and second, electricity being confined to suburban and interurban short hauls between large centers of population, the additional cost of avoiding heavy grades is comparatively small, and furthermore, if they are not avoided the loss in time would result in a correspondingly larger loss of revenue.

Under (*b*) "Construction" we find that items 11 and 12 apply to electric traction only, while there are no items applying to steam-locomotive traction only. These items might have been placed under the heading "equipment," as part of the things necessary in electric traction to replace steam locomotives and which could be added to motor cars or electric locomotives as equipment for an electric road. This phase of the question will be discussed under the heading "equipment." The engineering of an electric line might cost more than that of a steam road, but not necessarily; if the chief engineer was a competent railroad man with a thorough knowledge of electrical engineering, the cost would be the same; otherwise it would be increased by that amount paid an assistant engineer with special electrical knowledge, unless the line was costly enough to warrant several assistant engineers, one of whom could be versed in electrical matters.

The cost of surveying work, consisting of reconnaissance, exploration line ("shoo-fly"), preliminary, location, and their modifications or extensions, depending upon the importance of the line, its cost,

Mr. Davis.

EXPLANATION

Curve.		Multiplier for Vertical Scale.
1.	Gross Traffic Earnings, in Dollars	100 000
2.	Total Interest on Bonded Debt, in Dollars	10 000
3.	" Dividends on Capital Stock, in Dollars	10 000
4.	Gross Traffic Earnings per Mile of Track, in Dollars	100
5.	Percentage of Expenses to Gross Earnings	1
6.	" " Interest paid on Bonded Debt	1/10
7.	" " Dividends " " Capital Stock	1/10
8.	Gross Earnings per Revenue Train-Mile (Car-Mile), in cents	1
9.	" Expenses " " " " " " " "	1

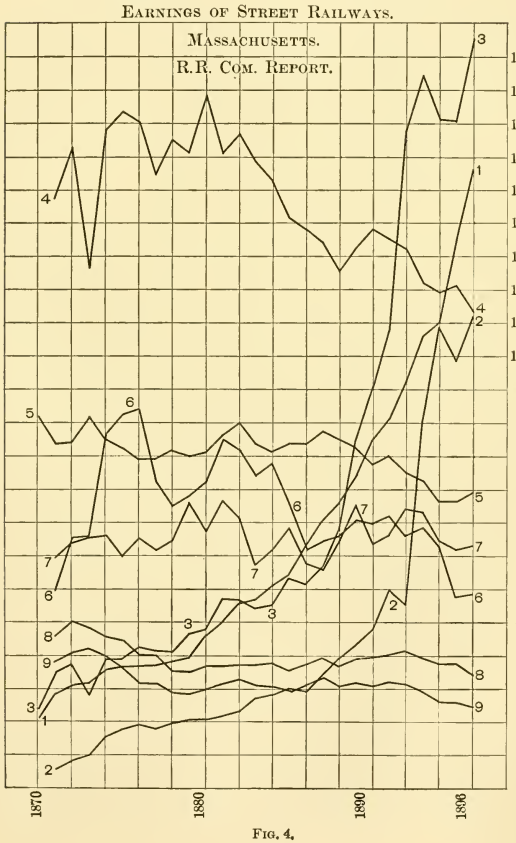

FIG. 4.

Mr. Davis. character of city and country, etc., would be the same in either of the cases under discussion for any specific example. The cost of clearing, grading, ditching, tunnels, masonry, culverts, fencing, cattle-guards, and those parts of the system pertaining to these general items, is not affected by the choice of system. It might be thought, at first, that the cost of masonry would be affected if lighter bridges and trestles could be used with electric traction, but the weight and size of masonry abutments, piers, footings, etc., would hardly be affected by any slight changes in the moving load, for they are much more dependent upon the weight of the earth embankment which they support and the character of the ground upon which they are built, together with their height and length. If they perform all their other functions according to the best engineering practice, they will usually be found far more substantial than, theoretically, they need to be, in order to uphold the weight of the superstructure. Road crossings will be the same; (the additional cost of connecting a "broken" third-rail system being included under item 12.)

In the design of bridges and trestles, the principal elements which determine their cost are the span, dead weight, live uniform-load, concentrated live-load at head of trains (locomotive and tender), the speed of trains, and the "hammer-blow" of the reciprocating parts of a steam locomotive, especially when it coincides with the period of oscillation of a bridge (very objectionable on bridges which are too light). Electric motors, either geared to the axles or gearless (concentric with the axle), have a rotative motion, and therefore do not produce the "hammer-blow." Owing to the peculiar construction of the electric locomotive or motor car, a greater proportion of the total weight is on the driving wheels than is the case with steam locomotives; therefore, with the same tractive effect (weight on drivers), the use of electric locomotives or motor cars reduces the moving weight at the head of the train, and the weight of the tender is saved by the use of electricity. These three facts, theoretically, would enable bridges and trestles to be built lighter for roads using electric motors, providing the loads were not concentrated upon a smaller wheel base, were it not for the fact that the necessity of providing for constantly increasing weights practically offsets the theoretical saving which might be made. If motors were used on every car of the train, such structures might be designed so as to be lighter and less costly, but, for reasons stated, this is unlikely. In addition, if freight service should continue to be handled by locomotives on the same tracks or adjoining ones, lighter bridges would not be possible.

The cost of ballasting, ties, rails and track-laying, lining and surfacing will be the same in either case (drilling rails for bonding is covered by item 11), although a small saving in maintenance of these items is possible for electric traction (discussed under "total

Mr. Davis.

Curve.	EXPLANATION.	Multiplier for Vertical Scale.
1.	Gross Traffic Earnings, in Dollars	1 000 000
2.	Total Interest on Bonded Debt, in Dollars	100 000
3.	" Dividends on Capital Stock, in Dollars	100 000
4.	Gross Traffic Earnings per Mile of Track, in Dollars	100
5.	Percentage of Expenses to Gross Earnings	1
6.	" " Interest Paid on Bonded Debt	1/10
7.	" " Dividends " " Capital Stock	1/10
8.	Gross Earnings per Revenue Train-Mile, in Cents	1
9.	" Expenses " " " " "	1
10.	Percentage of Passenger Earnings to Gross Earnings	½
11.	" " Freight " " " "	½
12.	" " Interest and Dividends paid on Capital Stock and Bonded Debt	1/10

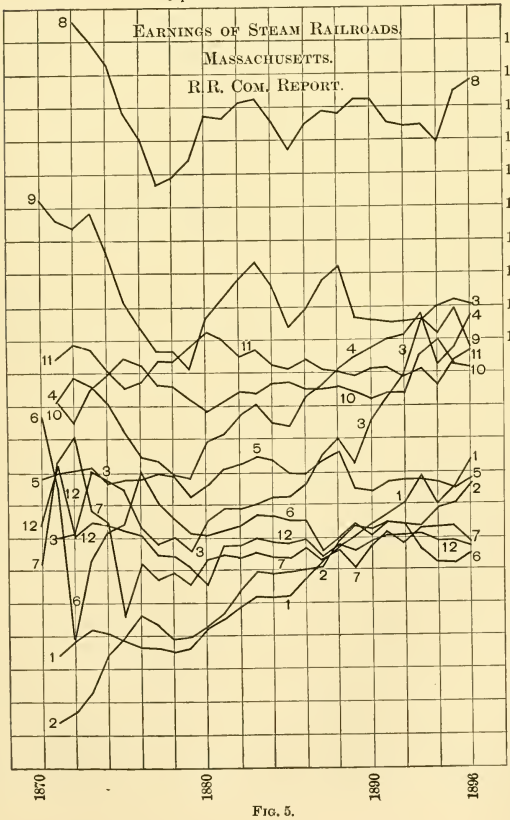

FIG. 5.

Mr. Davis. expense"), which indicates that the first cost could be reduced for the same cost of operating; yet, as this would be poor economy, the quality, weight and general character should be the same in either case. By the use of electricity it might, at first, appear that the signal and telegraph system would be reduced in first cost, but the only possible saving would be in the source of the current used, which forms so small a percentage of the whole that for our purposes any difference can be neglected.

Stations would cost the same under similar conditions, whether steam or electricity were used, although the electric road requires buildings for its centralized motive-power plant (that is, all but the transmission line and motors) which are not a burden on the steam road, yet these are no doubt balanced by round houses, coal stations, water stations and the larger machine shops required for steam locomotive repairs. Terminals will cost the same, whichever system is used.

So far, we have found that the first cost of construction is substantially the same, whether steam locomotives or electric motors are used, but, if the latter are adopted, some additional work is necessary, namely, (11) rail bonding and (12) overhead-trolley line or third rail. It will at once be seen that these items add considerably to the cost of construction of an electric road, as compared with that of a road operated by steam locomotives.

As an example, which will show, in a general way, the increased cost of electric-traction construction, let us assume, without any pretension to exactness, for each individual case will differ in the amount of increased cost, that the line is single track, one mile long; average speed of trains (including stops) 20 miles per hour; headway, one minute; length, six cars (one being a motor car); total weight of train, 200 tons when loaded to ultimate capacity; rails, 90 lbs. per yard; average nominal horse-power, 300 per train (will exert 450 H.-P., or even more, if required for short periods); total maximum average nominal horse-power per mile of track (one way) at above headway, 900; average loss in transmission, 20%; assumed average nominal horse-power at station, 1 125 (allowing for loss and including surplus power), located at one end of the line. With these assumptions, we have the following approximate additional cost for construction, if electricity is used. The electric return circuit would consist of the two lines of rail properly bonded with copper wires, two 4/0 B. & S. G. wires to each rail joint, which would cost (copper, 15 cents per pound), complete in place, including terminals and drilling rails, 70 cents each bond, or a total of $497. To this must be added the cross-bonding of the lines of rails, which varies much in practice. For our case we assume that this is unnecessary, if the bonds at the joints are properly made, and maintained with the care displayed on other parts of the system. The overhead-trolley line would be of iron

Mr. Davis.

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Passenger Train-Miles (Car-Miles.)	1 000 000
2.	Total Passengers Carried	2 000 000
4.	Gross Passenger Earnings. (Same as Curve 1, Fig. 4).	
6.	Receipts per Passenger Carried, in Cents.	1/10
7.	" " " Train-Mile. (Same as Curve 8, Fig. 4).	
8.	" " Mile of Track. (Same as Curve 4, Fig. 4).	
9.	Number of Passengers Carried per Mile of Track.	2 000
11.	Passenger Rides per Capita per Annum	1
12.	Receipts per Capita per Annum, in Dollars	1/10

PASSENGER TRAFFIC OF STREET RAILWAYS.

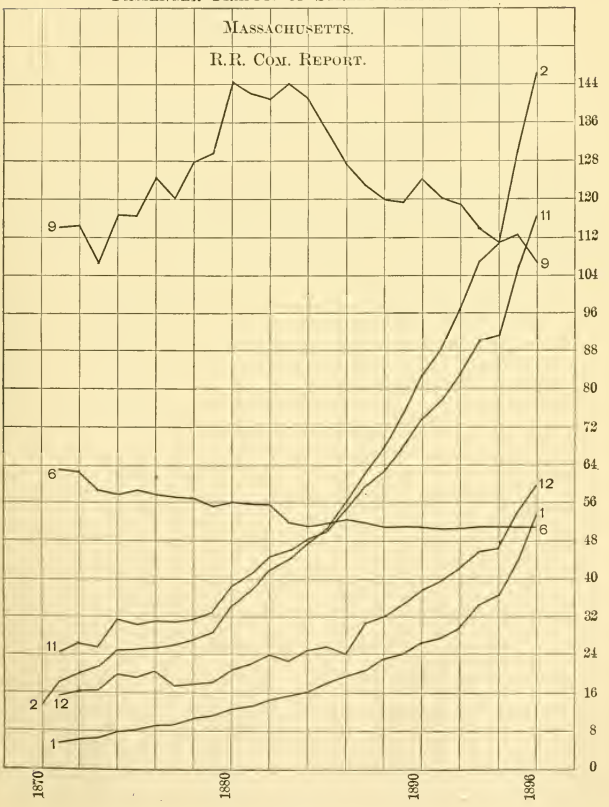

FIG. 6.

Mr. Davis. poles, center-construction (assuming that there would be a double track); trolley wire 423 200 circular mils; poles set every 100 ft., each weighing 1 300 lbs. (30 ft. by 8 ins. and 7 ins. and 6 ins.) set in concrete, would cost for one mile, approximately, $3 237 (one-half of poles with brackets and feeder supports, etc., set at $50 each; trolley 6 748 lbs., at 15 cents; special line material, $500; labor, $400).

To recapitulate: One mile of track equipped electrically will cost:

(11) Rail bonding. In this case the cost of ground-return circuit is not an extra cost against the electric road, for the rails are used, they having sufficient capacity to carry the current. (See Equipment)	$497
(12) Overhead-trolley line	3 237
Total extra per mile for electric trolley railroad	$3 734

If a third-rail system were used, which is probable where the road is on its own right-of-way and where high speeds and frequent service are necessary together with longer and heavier trains than the single cars of a railway system, then the increased cost would be approximately:

(11) Rail bonding, as before	$497
(12) 70.7 tons 90-lb. third-rail, special section, at $30	2 121
528 treated wooden supports at $1	528
176 joints to bond, at $3	528
Laying, at 5 cents per foot	264
Total extra per mile for third-rail electric railroad	$3 938

The apparent difference in favor of the overhead trolley must not be assumed to exist generally; accidentally, it happens to be less in the first case than in the second, while usually, on long lines with dense traffic, the reverse would be true.

There being no items applicable exclusively to steam railroads, to balance (11) and (12), it is seen that the first cost of construction of an electric road will be greater and that this increase will amount to several thousand dollars per mile of track, depending upon the conditions existing in each case.

Under (*c*) "Equipment," if the road is to be equipped to operate freight trains by electric power, there are two possible ways of doing so; first, by using the present style of freight cars and hauling them by electric locomotives; or second, by equipping each car with motors, thus making each an independent unit. The latter would only be feasible could the units be coupled together and controlled at the

Mr. Davis.

EXPLANATION.

Curve.		Multiplier for Vertical Scale.
1.	Passenger Train-Miles	1 000 000
2.	Total Passengers Carried	1 000 000
3.	Passengers Carried one Mile	10 000 000
4.	Gross Passenger Earnings, in Dollars	1 000 000
5.	Receipts per Passenger per Mile Carried, in Cents	1/10
6.	" " " Carried, in Cents	1
7.	" " " Train-Mile, in Cents	1
8.	" " Mile of Track, in Dollars	100
9.	Number of Passengers Carried per Mile of Track	100
10.	Miles Traveled per Passenger	1/4
11.	Passenger Rides per Capita per Annum	1
12.	Receipts per Capita per Annum, in Dollars	1/10

FIG. 7.

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EXPLANATION OF FIG. 8.

Curve.	Multiplier for Vertical Scale.
Curves Deduced from Fig. 2.	
1. Total Miles of Track	100
2. Annual Increase of Mileage	100
3. Total Capital Stock, in Dollars	500 000
4. " Bonded Debt, " "	500 000
5. " Capitalization, " "	1 000 000
6. " " per Mile of Track Owned, in dollars	500
Curves Deduced from Fig. 4.	
1. Gross Traffic Earnings, in Dollars	1 000 000
2. Total Interest on Bonded Debt, in Dollars	100 000
3. " Dividends on Capital Stock, in Dollars	100 000
4. Gross Traffic Earnings per Mile of Railroad, in Dollars	100
5. Percentage of Expenses to Gross Earnings	1
6. " " Interest Paid on Bonded Debt	$\frac{1}{10}$
7. " " Dividends Paid on Capital Stock	$\frac{1}{10}$
8. Gross Earnings per Revenue Train-Mile (Car-Mile) in Cents	1
9. " Expenses " " " " " " " "	1
Curves Deduced from Fig. 6.	
1. Passenger Train-Miles (Car-Miles)	1 000 000
2. Total Passengers Carried	10 000 000
4. Gross Passenger Earnings (Same as Curve 1, Fig. 4)	.....
6. Receipts per Passenger Carried, in Cents	$\frac{1}{10}$
7. " " " Train-Mile (Same as Curve 8, Fig. 4)	.....
8. " " Mile of Railroad (Same as Curve 4, Fig. 4)	.....
9. Number of Passengers Carried per Mile of Railroad	10 000
11. Passenger Rides per Capita per Annum	1
12. Receipts per Capita per Annum, in Dollars	$\frac{1}{10}$

head of the train; but imagine interchanging and making up a train of 50 to 100 cars arranged in this way by any present known mode of electric equipment, control and propulsion; no arguments seem necessary to show the folly of attempting it to-day. The only feasible method is evidently by electric locomotives at the head of such trains. The best practical example of this mode of operation is given in the Baltimore Tunnel of the Baltimore and Ohio Railroad, where 100-ton electric locomotives haul freight and passenger trains, together with their steam locomotives. They do the work satisfactorily, but the writer has seen no claims advanced to show that they are economical; in fact, the only defence for their use is one of cleanliness and ventilation in the tunnel, which is more satisfactory to passenger traffic, and undoubtedly attracts and stimulates it. The installation cost, if the writer's memory serves, over $2 000 000 for a few miles of track; so that the interest on this sum is much more than enough to offset any saving in operation, if even the latter can be shown. This is extremely doubtful. Such an investment could not be justified on any grounds if only freight were handled, although it may be by an increase in the passenger traffic. Freight trains are slow-moving, heavy units, requiring large powers constantly applied, and are run at comparatively infrequent intervals. The effort of all steam railroad man-

Mr. Davis.

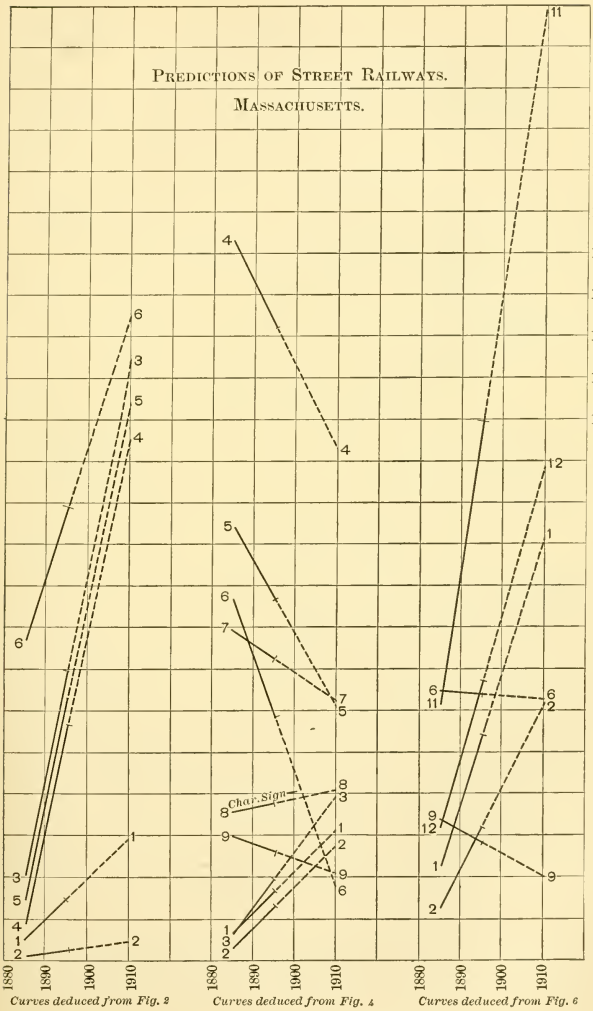

FIG. 8.

Mr. Davis.

EXPLANATION OF FIG. 9.

Curve.	Curves Deduced from Fig. 3.	Multiplier for Vertical Scale.
1.	Total Miles of Track	100
2.	Annual Increase of Mileage	10
3.	Total Capital Stock, in Dollars	10 000 000
4.	" Bonded Debt, " "	1 000 000
5.	" Capitalization, " "	10 000 000
6.	" " per mile of Railroad, in Dollars	1 000

	Curves Deduced from Fig. 5.	
1.	Gross Traffic Earnings, in Dollars	1 000 000
2.	Total Interest on Bonded Debt, in Dollars	100 000
3.	" Dividends on Capital Stock, in Dollars	100 000
4.	Gross Traffic Earnings per Mile of Railroad, in Dollars	100
5.	Percentage of Expenses to Gross Earnings	1
6.	" " Interest Paid on Bonded Debt	$\frac{1}{10}$
7.	" " Dividends " " Capital Stock	$\frac{1}{10}$
8.	Gross Earnings per Revenue Train-Mile, in Cents	1
9.	" Expenses " " " " "	1
10.	Percentage of Passenger Earnings to Gross Earnings	$\frac{1}{2}$
11.	" " Freight " " " "	$\frac{1}{2}$
12.	" " Interest and Dividends Paid on Capital Stock and Bonded Debt	$\frac{1}{10}$

	Curves Deduced from Fig. 7.	
1.	Passenger Train-Miles	1 000 000
2.	Total Passengers Carried	10 000 000
3.	Passengers Carried One Mile	100 000 000
4.	Gross Passenger Earnings, in Dollars	1 000 000
5.	Receipts per Passenger per Mile Carried, in Cents	$\frac{1}{10}$
6.	" " " Carried, in Cents	1
7.	" " " Train-Mile, in Cents	1
8.	" " Mile of Railroad, in Dollars	100
9.	Number of Passengers Carried per Mile of Railroad	1 000
10.	Miles Traveled per Passenger	$\frac{1}{2}$
11.	Passenger Rides per Capita per Annum	1
12.	Receipts per Capita, per Annum, in Dollars	1

agers is to increase train-loads, so as to decrease expenses per train-mile. If we apply the following argument and figures, changing the example to moving heavy freight trains, it will be seen at once that such traffic cannot now be handled by electric locomotives, involving heavy line work and large central stations (idle most of the 24 hours), for the main reason that interest on first cost will more than offset the largest possible saving in expenses. This saving would not exist, but if it should, more than 50% would be needed.

Passenger cars would cost the same in either case, as the cost of installing electric lighting and heating systems in the cars would balance the present gas lighting and steam heating systems of railroad passenger cars.

Should motor cars be used, carrying passengers, there would be a saving equal to the cost of one passenger car per train in favor of the electric road. If an electric locomotive were used, there would be no such saving. If every car were a motor car, then the cost of a steam

Mr. Davis.

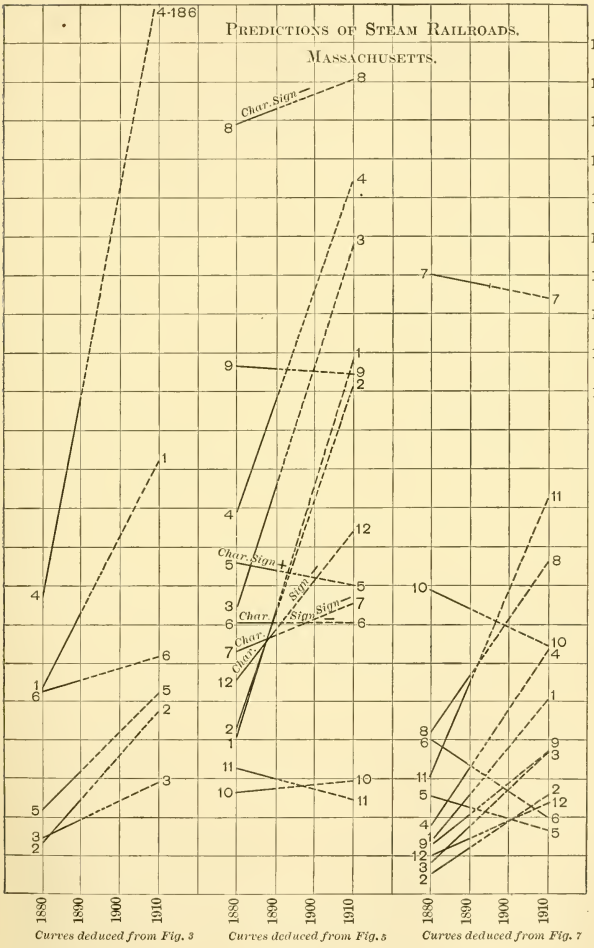

FIG. 9.

Mr. Davis. locomotive per train would be saved in the case of electric traction; this, however, would add to the cost of each car. This method is not satisfactorily solved to-day, although short trains are operated with more or less success.

Steam locomotives and tenders are to steam roads what motor cars (or electric locomotives), electric-feeder lines and ground-return circuits, and central power stations (without buildings) are to electric railways. To show the relative costs, let us again consider the former example. The feeder line would be about two 4/0 B. & S. G. wires of bare copper, as the company owns the right-of-way and crossings of public highways at grade are unlikely on roads of the character we are discussing; these would cost, in place, $1 117 ($1 012 for copper and $105 for stringing). It will be noticed that the trolley wire suggested is much larger than that used in general practice, and, as it has large carrying capacity, part of its cost might have been included in the cost of feeders. If the third rail were used in place of an overhead trolley, as explained before, this feeder system would be unnecessary, as the 90-lb. third rail when properly bonded has enough section to carry the current.

In our example the area of copper in the overhead system is 846 400 circular mils, equivalent to 920 mils diameter, which gives a total cross-section of 0.657 sq. in. A 90-lb. rail has a sectional area of 9 sq. ins. Taking the relative conductivity of steel and copper as 1 to 6, we find that each rail is equivalent to 1.5 sq. ins. of copper, and both rails to 3 sq. ins.; therefore, until the overhead system had a cross-section of more than 3 sq. ins., there would be no necessity for any copper ground-return circuit, so long as the bonding section at the joints was the same as that of the overhead work. In case the area of both rails divided by 6 is less than the area of overhead copper, the copper ground-return wire should be of a section equivalent to the excess in section of overhead copper. This ground return can be run on the pole line and connected down to the rails at intervals, or along the track, as may seem best in any given case.

The central power station of 1 125 nominal H.-P. could be erected complete (without buildings, included under "construction") for $50 per H.-P., which would provide a plant of the most modern kind, giving the greatest economy (in the largest sense). In general, it would consist of a steel stack lined with fire-brick, water-tube boilers, pumps, piping, condensers, heaters, blowers, direct-connected Corliss engines, direct-connected generators, slate switchboards, best indicating and recording instruments, etc., with all the best labor-saving and automatic devices which help to make a thoroughly successful plant. The total cost of the power plant would be $56 250. We will not discuss the use of an electric locomotive, but assume the use of motor cars drawing trailers, as it would only make a small difference

in the total amount given in the example. The figures, in any case, are only indicative. The same can be said of equipping each car as a motor car. Mr. Davis.

In the estimate of 200 tons as the weight of a 6-car train, one being a motor car, the car bodies assumed are 45 ft. over sills and 51 ft. over all, with two trucks having four wheels each, the wheels being 33 ins. in diameter; car body weighing 22 000 lbs.; trucks, 13 000 lbs.; total, 35 000 lbs.; average maximum passengers, 150 at 150 lbs. each, 22 500 lbs.; making the total weight of the passenger car loaded 57 500 lbs. The motor car is assumed to have four motors, one on each axle or geared to it, its total weight loaded being 112 500 lbs. (56.25 tons loaded, 45 tons empty, all on drivers). The trucks of this motor car would be heavier than those of the trail cars, and with larger wheels. With two four-wheel pivotal trucks, the weight on each driving wheel would be 14 082 lbs. loaded, or 11 250 lbs. empty. The complete trail cars can be assumed to cost, for steam or electricity, about $3 500; the complete motor car, about $7 000. A steam locomotive and tender of the same power and tractive force would cost about $9 000 (engine, $7 700; tender, $1 300). It will be noticed that the heating of cars has had no consideration herein. While this would add to the first cost of an electric system, by increasing the required capacity of transmission line and power house, and would undoubtedly add to the cost of operating as compared with steam-locomotive practice, yet available data are so unreliable that a discussion of them is postponed. To recapitulate: On the basis of our example, we see that items 2, 5 and 6 of the electric system of equipment leave only item 1 on the steam side of the ledger to balance them. It equalizes a small part only. The New York Central Railroad, in 1885, with 2 716 miles of single track, had $2 337 (5.7%) per mile invested in locomotives for passenger and freight service, and must have considerably more today. If we deduct investment for freight locomotives and compare with the following, it gives some idea of the difference in first cost of equipment against the electric system with only passenger service considered:

Feeder line	$1 117
Ground return (in this case no additional cost)	.....
Central power station	56 250
	$57 367
Deduct difference between 3 motor cars and 3 locomotives and tenders	6 000
Total extra per mile for electric trolley road	$51 367

To estimate a 10-mile road at ten times the above would be unfair, for, unless the train density remained the same on every mile and a power station were built for every mile, it could not be in direct pro-

Mr. Davis. portion. In general, the feeder system increases in cost with the distance, while the power station increases in cost with the weights moved and their speed. There would undoubtedly be found conditions of very light trains, at very frequent intervals, over short distances, where the larger cost of locomotives and tenders would balance the items which usually make electricity the more costly; these conditions must more nearly approach those which now exist in street railway service. It is evident that with units weighing 10 to 15 tons, run at intervals of 20 seconds to 5 minutes, any steam-locomotive system would prove undesirable, both by its greater first cost and operating expenses.

"Electric feeder lines and ground-return circuit" is placed under the heading "equipment" rather than under "construction" for the purposes of this discussion; in a system operated by electricity the opposite course would be pursued, and has been followed in considering "total expenses." "Central power stations" (all but buildings) would naturally be in the motive power department, and are so considered herein.

"General" items of "first cost" would be unaffected by the character of the system. A few years ago, when the word "electricity," as applied to street railways, was more attractive in banking and investing circles, higher prices for bonds might have been obtained by the use of that system, but elements of this kind are not within the scope of this discussion. Taxes are based on value, not on cost, and are therefore unaffected by any particular system.

Should an existing railroad change its motive power on all or a part of its system, the following items would represent the investment which would be made, the interest upon which must either be saved or earned to warrant it:

(*b*) Construction.

1. Engineering.
(Electric) 11. Rail bonding.
(Electric) 12. Overhead-trolley line or third rail.
15. Buildings for central power stations.

(*c*) Equipment.

(Electric) 2. Motor cars or electric locomotives.
3. Passenger cars (trail cars, or if motors are applied to every car).
(Electric) 5. Electric feeder lines and ground-return circuit.
(Electric) 6. Central power stations (not including buildings which are considered under "construction").

(*d*) General.

1. Discount on bonds.
2. Interest on bonds to opening of road.
3. Taxes to opening of road.
5. Contingent and miscellaneous, not itemized, to opening of road.

TABLE No. 2.—COST OF NEW YORK CENTRAL AND HUDSON RIVER RAILROAD, 1885.

(2 716.05 miles of single track.)

Item.	Cost per mile of single track.	Percentage.
1. Grading and masonry	$7 749	18.9
2. Bridges, etc.	1 066	2.6
3. Superstructure	11 439	27.9
4. Stations, etc.	5 453	13.3
5. Land and land damages	5 576	13.6
6. Locomotives	2 337	5.7
7. Passenger cars	574	1.4
8. Freight cars	5 576	13.6
9. Engineering	1 107	2.7
10. Floating equipment	123	0.3
Total	$41 000 (approx.)	100.0
11. Stock	$32 900	
12. Bonds	20 700	
Total capitalization	$53 600	

All other items would be unaffected on an existing system. Taking them up in detail:

Engineering expenses would not be as great as in the case of a new line. Items 11 and 12 under (*b*) "Construction," are applicable to this case, as already discussed for new lines. Buildings for power stations would be an additional cost, unless steam locomotives were entirely discarded and old round houses utilized; this, however, is hardly likely. Under practically all circumstances steam locomotives and tenders would be utilized on other parts of the system or for freight service, and therefore no allowance need be made for abandoning them. Items 2 and 3 under (*c*) "Equipment" should be new, and old equipment used elsewhere. Any attempt to use existing trucks or car bodies, either for motor cars or trailers, would be as bad a mistake as was made in the early days of the change from horse to electric traction on street railways. Items 5 and 6, under the same heading, are also applicable to this case, as already discussed for new lines. Items 1, 2, 3 and 5, under (*d*) "General" would have to be charged against the change, in proportion to the cost of the same. For all practical purposes, the total cost of a change of system is seen to be about equal to the difference in cost of two new lines, as follows:

(*b*) Construction	$3 734	per mile of track.
(*c*) Equipment	51 367	" "
(*d*) General (15%)	8 265	" "
Total	$63 366	" "

Mr. Davis.

TABLE No. 3.—ELECTRIC SUBURBAN AND

NAME OF RAILWAY.	Index No. of Ry.	Total passengers carried per annum; year ending June 30th, 1898.	Total length of single track operated (miles).	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.	Total car-miles operated per annum.
MAINE (19 roads).						
Augusta, Hallowell & Gardiner Railroad	S 1	816 115	7.00	53	116 588	201 534
Bangor, Orono & Old Town Railroad	S 2	1 225 028	17.40	88	73 414	408 570
Biddeford & Saco Ry	S 3	337 860	5.72	67	58 984	136 312
Lewiston & Auburn Horse R. R	S 4	1 160 445	14.00	77	82 888	350 000(a)
Portland & Cape Elizabeth Ry	S 5	1 095 666	11.52	67	90 600	128 945
Portsmouth, Kittery & York St. Ry	S 6	812 220	15.20	58	54 748	250 000(a)
Rockland, Thomaston & Camden St. Ry	S 7	960 578	16.64	62	57 727	268 609
Totals and averages	7	6 407 912	87.48	74	73 249	1 743 970
CONNECTICUT (31 roads).						
Danbury & Bethel St. Ry	S 8	823 669	10.58	60	77 851	244 533
Derby Street Ry	S 9	712 344	5.89	56	120 958	154 275
Fair Haven & Westville R. R	S 10	4 898 563	19.89	56	246 272	1 283 642
Hartford, Manchester & Rockville Tramway	S 11	599 429	17.88	63	33 525	243 542
Hartford & West Hartford Horse R. R	S 12	375 375	9.70	98	38 698	188 892
Meriden Electric R. R	S 13	1 439 285	17.00	77	84 663	445 864
Norwalk Street Ry	S 14	748 842	7.52	69	99 580	167 592
Norwalk Tramway	S 15	816 113	17.52	78	46 466	302 608
Norwich Street Ry	S 16	1 256 626	11.91	70	105 476	223 347
Torrington & Winchester St. Ry	S 17	436 726	12.56	64	39 548	162 684
Bristol & Plainville Tramway	S 18	543 032	7.34	84	73 978	145 629
Central Railway & Electric (New Britain)	S 19	1 434 105	16.55	75	86 713	338 403
Totals and averages	12	14 143 909	154.34	67	91 641	3 901 011
MASSACHUSETTS (103 roads).						
Arlington & Winchester	S 20	330 594	3.33	67	99 277	79 210
Athol & Orange	S 21	602 227	6.88	63	87 533	114 075
Braintree & Weymouth	S 22	992 063	11.88	65	83 507	267 335
Bridgewater, Whitman & Rockland	S 23	546 485	12.68	75	43 098	220 363
Brockton, Bridgewater & Taunton	S 24	1 353 082	21.07	58	64 200	490 006
Brockton & East Bridgewater	S 25	364 512	9.22	75	39 531	105 528
Dartmouth & Westport	S 26	551 674	13.73	70	40 163	337 232
Dighton, Somerset & Swansea	S 27	775 266	16.78	71	46 188	282 591
Fitchburg & Leominster	S 28	2 178 863	21.43	64	101 673	671 374
Gloucester, Essex & Beverly	S 29	1 556 243	22.71	61	50 911	355 932
Greenfield & Turners Falls	S 30	712 647	12.86	64	55 416	178 076
Haverhill & Amesbury	S 31	1 408 722	26.04	65	54 098	433 833
Haverhill, Georgetown & Danvers	S 32	477 268	6.13	75	77 858	143 160
Hingham	S 33	907 648	19.38	74	46 801	291 842
Hoosac Valley (North Adams)	S 34	1 536 505	13.00	71	118 138	349 752
Interstate Consolidated (R. I.)	S 35	2 589 797	22.42	68	115 513	660 306
Leominster & Clinton	S 36	856 817	11.46	58	74 733	248 306
Lowell, Lawrence & Haverhill	S 37	8 981 702	64.82	56	138 572	1 741 291
Lowell & Suburban	S 38	7 679 147	64.77	61	118 545	1 938 352
Lynn & Boston	S 39	29 063 234	153.83	57	188 928	5 800 287
Marlborough	S 40	756 465	7.54	90	100 314	228 834

(a) Estimated.

INTERURBAN RAILWAYS (R. R. COM. REPORT). Mr. Davis.

PASSENGER EARNINGS.								
Per mile of single track operated.	Per car-mile operated (cents).	Per passenger per trip (cents).	Total per annum from passengers; year ending June 30th, 1898.	No. of passengers carried per car-mile operated.	Percentage of dividends paid; year ending June 30th, 1898.	Permanent investment per mile of single track owned; year ending June 30th, 1898.	Capital stock and debts per mile of single track owned; year ending June 30th, 1898.	Total operating expenses per car-mile operated (cents).
$5 802	20.15	4.97	$40 227	4.04	4.0	$26 982	$39 687	10.90
3 659	15.58	5.19	61 251	2.99	8.0	15 208	16 475	13.76
4 363	18.30	7.38	24 837	2.47	....	23 483	27 651	12.27
3 721	14.80(*a*)	4.48	51 815	3.31 (*a*)	....	29 605	31 985	11.51 (*a*)
3 795	43.31	5.09	54 782	8.49	1.0	52 977	55 637	28.81
2 736	16.42(*a*)	5.03	41 061	3.25 (*a*)	1.2	26 556	26 788	9.57 (*a*)
4 099	25.39	7.10	48 028	3.02	....	19 058	31 631	15.93
$3 680	18.46	5.02	$322 001	3.10	....	$28 839	$25 409	13.71
4 045	17.51	5.13	42 826	3.37	4.0	50 844	47 399	10.64
6 024	23.00	4.98	35 486	4.62	....	55 515	58 527	13.02
12 459	19.31	5.06	247 812	3.82	6.0	36 436	43 298	10.87
3 422	25.13	10.20	61 192	2.46	....	29 317	28 002	15.92
3 051	15.67	7.88	29 599	1.99	....	58 323	52 046	15.49
4 245	16.19	5.08	72 176	3.23	....	94 373	92 507	12.63
4 844	21.74	4.86	36 430	4.47	4.5	31 338	32 159	15.09
2 099	12.16	4.50	36 790	2.70	2.0	38 927	33 466	9.58
4 529	24.17	4.29	53 973	5.63	....	45 498	42 511	17.08
1 927	14.88	4.86	24 202	3.05	....	29 987	28 933	9.57
3 641	18.34	4.90	26 718	3.73	3.0	35 827(*b*)	31 460(*b*)	15.49
3 856	18.86	4.45	63 821	4.24	....	78 934(*b*)	65 123(*b*)	14.26
4 736	16.17	5.16	$731 025	3.62	....	$49 553	$50 229	12.53
4 938	20.76	4.97	16 444	4.17	....	29 831	28 928	13.86
4 281	25.82	4.89	29 339	5.27	8.0	19 771	17 464	16.21
4 087	18.16	4.89	48 258	3.71	3.0	21 248	20 691	11.81
2 136	12.29	4.96	26 960	2.47	....	16 577	16 940	9.26
3 381	14.55	5.27	69 484	2.76	3.0	24 903	24 405	8.29
1 988	17.46	5.06	18 352	3.45	....	20 981	20 341	13.11
7 055	28.74	17.57	95 578	1.63	8.0	26 594	24 541	20.18
3 668	21.79	7.94	61 516	2.75	....	27 541	27 328	15.56
6 267	20.01	6.16	134 014	3.24	8.0	25 905	25 787	12.84
3 599	16.59	5.11	58 698	3.24	3.0	17 837	17 880	10.23
2 689	19.38	4.85	33 677	4.00	4.0	16 835	16 386	12.37
3 443	20.67	6.36	88 060	3.24	....	25 993	25 811	13.46
3 914	16.76	5.03	23 757	3.33	....	18 176	17 096	12.63
2 304	15.31	4.92	44 197	3.75	....	22 059	22 741	11.30
5 894	21.92	4.99	75 214	4.38	6.0	19 558	17 881	15.64
6 039	20.50	5.23	134 714	3.99	6.0	20 465	25 083	14.02
3 805	17.57	5.09	43 165	3.45	....	31 282	32 487	10.22
6 980	25.98	5.04	448 938	5.15	4.0	48 795	47 401	14.68
5 986	20.00	5.05	380 177	3.96	3.0	36 004	36 169	12.28
9 648	25.59	5.11	1 472 154	5.01	8.0	58 851	59 200	14.75
5 117	17.24	5.10	85 952	3.38	....	24 720	26 967	15.60

(*b*) Includes cost of electric lighting plant, which could not be separated.

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TABLE No. 3 (*Concluded*).—ELECTRIC SUBURBAN AND

NAME OF RAILWAY.	Index No. of Ry.	Total passengers carried per annum; year ending June 30th, 1896.	Total length of single track operated (miles).	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.	Total car-miles operated per annum.
Milford, Holliston & Framingham	S 41	1 668 831	20.34	60	82 051	465 991
Natick & Cochituate	S 42	1 089 877	11.00	84	99 080	257 772
Newburyport & Amesbury	S 43	1 139 652	18.58	92	82 015	320 056
Newton	S 44	1 827 120	15.78	67	115 743	465 458
Newton & Boston	S 45	1 098 486	13.82	71	79 468	456 530
Norfolk Central (Dedham)	S 46	712 551	6.52	72	109 337	185 583
Norfolk Suburban (Hyde Park)	S 47	1 972 521	10.95	77	180 188	423 986
Northampton	S 48	1 812 627	17.12	62	105 847	487 369
North Woburn	S 49	809 869	7.80	70	103 803	189 073
Providence & Taunton	S 50	673 714	14.92	36	45 149	99 983
Quincy & Boston	S 51	2 712 342	30.06	64	90 240	564 141
Reading & Lowell	S 52	332 530	11.84	93	28 085	146 373
Rockland & Abington	S 53	1 521 581	17.32	83	88 336	397 812
Rockport	S 54	567 164	8.16	69	69 505	146 377
Southbridge & Sturbridge	S 55	552 886	7.75	68	71 320	149 588
South Middlesex (Natick)	S 56	1 134 670	13.07	63	86 815	269 181
Taunton & Brockton	S 57	949 244	17.23	76	55 093	249 362
Wakefield & Stoneham	S 58	1 129 681	15.30	58	73 835	374 431
Warren, Brookfield & Spencer	S 59	966 236	19.22	61	50 275	288 827
Worcester & Blackstone Valley	S 60	281 043	6.92	82	40 665	110 098
Worcester & Marlborough	S 61	1 229 738	17.36	65	70 837	342 877
Worcester & Suburban	S 62	3 275 611	21.91	68	149 483	714 529
Totals and averages	43	91 288 034	864.63	64	105 567	22 037 982
(*c*) *Totals and Averages of Table No. 3.*						
Maine	(*d*) 7	6 407 912	87.48	74	73 249	1 743 970
Connecticut	(*d*)12	14 143 909	154.34	67	91 641	3 901 011
Massachusetts	(*d*)43	91 288 034	864.63	64	105 567	22 037 982
Totals and averages	(*d*)62	111 839 855	1106.45	65	101 079	27 682 963

(*c*) Railways in Rhode Island, New Hampshire and Vermont are not treated in this table.

It should be remembered that the figures at the foot of page 875 are not exact, nor would it be fair to multiply by the number of miles of track equipped.

It is evident that, whether a new line be built under the same conditions of existing steam railroads, or an old system be changed over, the first cost will be against electric motive power; and to overcome this disadvantage the operating expenses must be reduced sufficiently to more than take care of the interest on this additional cost; or else the change must create an increased traffic which justifies it.

Wellington* gives a table showing the distribution of first cost of the New York Central and Hudson River Railroad in 1885, from which Table No. 2 has been calculated.

* "Economic Theory of Railway Location," p. 71.

INTERURBAN RAILWAYS (R. R. COM. REPORT). Mr. Davis.

Passenger Earnings.									
Per mile of single track operated.	Per car-mile operated (cents).	Per passenger per trip (cents).	Total per annum from passengers; year ending June 30th, 1898.	No. of passengers carried per car-mile operated.	Percentage of dividends paid; year ending June 30th, 1898.	Permanent investment per mile of single track owned; year ending June 30th, 1898.	Capital stock and debts per mile of single track owned; year ending June 30th, 1898.	Total operating expenses per car-mile operated (cents).
$4 103	17.91	5.00	$88 283	3.58	....	$20 096	$19 351	10.73
5 044	21.53	5.09	55 070	4.22	6.0	11 463	10 580	18.21
3 259	18.71	5.26	56 982	3.56	....	29 847	31 166	17.16
6 069	20.58	5.24	91 465	3.92	8.0	33 830	32 239	13.83
4 729	14.32	5.95	54 322	2.40	5.0	40 179	39 545	10.20
5 437	19.09	4.97	35 351	3.88	....	26 919	19 487	13.81
9 079	23.44	5.04	98 998	4.65	7.0	25 322	24 939	18.05
5 391	18.94	5.09	89 466	3.71	8.0	22 965	21 778	11.82
5 234	21.60	5.04	40 690	4.28	....	26 302	27 622	15.13
2 382	35.56	5.28	35 456	6.73	....	30 302	28 821	12.86
4 515	24.06	5.00	128 857	4.80	3.5	22 225	20 806	15.53
1 413	11.44	5.03	16 588	2.27	....	13 074	13 685	10.63
4 521	19.58	5.12	75 315	3.80	6.0	16 103	15 937	16.57
3 484	19.42	5.01	28 358	3.87	6.0	16 888	15 602	13.52
3 505	18.03	5.04	37 177	3.02	5.0	16 116	15 842	12.74
4 370	21.22	5.03	56 517	4.21	6.0	19 797	19 189	13.39
2 824	19.51	5.13	48 510	3.80	6.0	16 323	16 149	14.78
4 056	16.58	5.49	61 838	3.01	5.0	25 291	25 163	9.59
2 510	16.17	4.99	48 041	3.34	....	17 662	17 204	10.23
2 033	12.78	5.00	14 070	2.55	....	12 867	12 671	10.44
3 780	19.14	5.34	64 787	3.58	6.0	26 690	25 453	12.48
7 044	21.60	4.71	152 402	4.58	4.0	37 364	38 688	14.63
$5 437	21.33	5.15	$4 702 191	4.14	....	$30 131	$30 507	13.74
Electric Suburban and Interurban Railways.								
$3 680	18.46	5.02	$322 001	3.10	....	$28 839	$25 409	13.71
4 736	16.17	5.16	731 025	3.62	....	49 553	50 229	12.53
5 437	21.33	5.15	4 702 191	4.14	....	30 131	30 507	13.74
$5 201	20.79	5.14	$5 755 217	4.04		$32 738	$32 855	13.20

(*d*) No. of roads taken in each State.

From Table No. 3 we find that the average cost per mile of single track of 62 suburban and interurban electric railways in New England is $32 738, while the highest is $58 851, and the lowest $11 463. Obviously, it would be unfair to compare the steam railroad chosen with the foregoing average, as the former must exceed the average of its own class by over 25 per cent. Adding this percentage to the foregoing average gives approximately $41 000 per mile of single track as a fair comparative cost for an electric interurban railway with the same standard of construction and equipment as the New York Central and Hudson River Railroad. The construction account of such railways is about 50% of the whole; the equipment 30% (including power stations); and lands, buildings and other permanent investment about 20 per cent. From the foregoing, Table No. 8 has been computed.

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TABLE No. 4.—ELECTRIC CITY

NAME OF RAILWAY.	Index No. of Ry.	Total passengers carried per annum; year ending June 30th, 1898	Total length of single track operated (miles).	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.	Total car-miles operated per annum.
MAINE (13 roads).						
Bangor St. Ry.	C 1	1 523 837	9,70	82	157 096	326 197
Bath St. Ry.	C 2	451 907	4,25	66	106 331	103 395
Calais St. Ry.	C 3	485 613	7,00	79	69 373	183 960
Portland Railroad	C 4	5 444 897	30,95	68	175 925	1 155 854
Totals and averages	4	7 906 254	51,90	72	152 336	1 769 406
CONNECTICUT (31 roads).						
Bridgeport Traction	C 5	4 108 260	52,60	51	78 107	1 454 638
Hartford Street Ry.	C 6	8 886 229	56,64	69	156 889	2 228 932
Middletown St. Ry.	C 7	401 948	7,30	73	55 061	102 801
New Haven St. Ry.	C 8	3 093 458	26,80	60	115 427	883 731
New London St. Ry.	C 9	666 844	6,96	74	95 810	137 841
Stamford St. Ry.	C 10	618 845	11,05	97	56 004	226 263
Winchester Ave. (New Haven)	C 11	4 381 462	17,54	61	249 798	764 801
Waterbury Traction	C 12	2 515 116	12,18	58	206 495	435 431
Totals and averages	8	24 672 162	191,07	63	129 131	6 234 438
MASSACHUSETTS (103 roads).						
Boston Elevated	C 13	181 321 295	278,35	71	651 410	32 209 150
Braintree	C 14	944 064	14,26	80	66 204	267 364
Brockton	C 15	6 787 425	43,37	63	156 511	1 463 110
Commonwealth Ave. (Newton)	C 16	1 447 822	12,73	92	113 724	478 017
Framingham Union	C 17	636 673	6,36	60	100 137	111 966
Gardner Electric	C 18	341 038	4,34	82	76 569	77 126
Globe (Fall River)	C 19	6 627 700	29,00	62	227 842	1 280 612
Gloucester	C 20	1 369 961	11,25	67	121 774	283 601
Holyoke	C 21	4 233 900	30,56	67	138 535	1 107 962
Pittsfield Electric	C 22	1 318 035	10,10	65	130 537	260 906
Plymouth & Kingston	C 23	691 048	8,75	67	78 977	181 650
Springfield	C 24	11 611 232	61,60	74	188 479	3 133 475
Taunton	C 25	1 269 804	17,13	54	74 115	383 711
Union (New Bedford)	C 26	3 776 878	19,96	64	189 222	799 738
Wellesley & Boston	C 27	709 887	4,66	67	152 336	220 980
West Roxbury & Roslindale	C 28	1 460 920	10,26	66	142 307	323 023
Worcester Consolidated	C 29	10 637 221	41,81	73	254 388	2 121 402
Woronoco	C 30	485 636	7,08	92	68 554	257 497
Totals and averages	18	235 670 539	611,66	72	385 296	44 961 284
Totals and averages (omitting C 13)	17	54 349 244	333,31	71	163 059	12 752 134
NEW HAMPSHIRE (5 roads).						
Union St. Ry.	C 31	592 567	7,50	96	79 008	178 740
Nashua St. Ry.	C 32	954 333	14,46	80	65 928	400 353
Concord St. Ry.	C 33	971 160	12,50	92	77 692	300 000 (*a*)
Manchester St. Ry.	C 34	3 290 386	17,62	68	201 124	783 635
Laconia St. Ry.	C 35	229 171	3,63	75	68 043	59 665
Totals and averages	5	6 037 617	55,11	77	109 555	1 701 393
(b) Totals and Averages of Table No. 4						
Maine	(*c*) 4	7 906 254	51,90	72	152 336	1 769 406
Connecticut	(*c*) 8	24 672 162	191,07	63	129 131	6 234 438
Massachusetts	(*c*) 17	54 349 244	333,31	71	163 059	12 752 134
New Hampshire	(*c*) 5	6 037 617	55,11	77	109 555	1 701 393
Totals and averages	(*c*)34	92 965 277	631,39	69	147 237	22 457 371

(*a*) Estimated. Given in report as "96 000 estimated," which is obviously wrong.
(*c*) No. of roads taken in each State.

RAILWAYS (R. R. COM. REPORTS). Mr. Davis.

PASSENGER EARNINGS.								
Per mile of single track operated.	Per car-mile operated (cents).	Per passenger per trip (cents).	Total per annum from passengers; year ending June 30, 1898.	No. of passengers carried per car-mile operated.	Percentage of dividends paid; year ending June 30, 1898.	Permanent investment per mile of single track owned; year ending June 30, 1898.	Capital stock and debts per mile of single track owned; year ending June 30, 1898.	Total operating expenses per car-mile operated (cents).
$6 731	20.00	4.28	$61 932	4.67	....	$58 864	$46 974	16.60
5 063	20.56	4.70	21 190	4.37	2.5	40 592	40 976	13.66
3 221	12.63	4.78	22 964	2.64	....	28 571	30 097	10.06
9 761	26.13	5.54	300 237	4.71	6.0	34 712	34 263	17.95
$7 825	22.97	5.15	$406 323	4.46	....	$35 441	$34 016	16.63
4 581	16.56	5.86	241 007	2.82	....	73 998	74 349	8.56
7 734	19.65	4.92	438 069	3.99	3.5	46 913	41 569	13.60
2 520	17.90	4.57	18 402	3.91	....	32 420	35 943	13.18
5 401	16.38	4.67	144 755	3.50	....	70 898	65 992	9.93
4 454	22.49	4.64	31 003	4.84	5.0	54 963	52 986	16.82
2 726	13.31	4.86	30 122	2.74	....	27 841	27 899	12.95
9 388	21.53	3.76	164 666	5.73	8.16	57 452	40 766	13.30
9 644	26.98	4.67	117 469	5.78	3.0	75 016	87 857	15.59
$6 204	19.01	4.80	$1 185 493	3.95	....	$59 129	$62 517	12.04
32 976	28.50	5.06	8 967 587	5.62	2.25	98 600	103 655	20.39
3 163	16.87	4.78	44 635	3.53	4.0	13 115	13 198	13.46
7 749	22.97	4.95	330 559	4.63	6.0	34 997	34 926	14.52
5 783	15.40	5.08	71 331	3.02	....	29 068	29 145	14.27
4 785	27.17	4.78	29 954	5.68	....	22 523	21 962	16.48
3 940	22.18	5.01	17 020	4.42	....	17 533	18 403	18.32
10 943	24.86	4.80	301 238	5.17	....	75 447	74 886	15 57
6 272	24.88	5.15	68 498	4.83	6.0	33 377	31 211	16 71
7 496	20.68	5.41	227 165	3.82	8.0	29 964	27 950	13.95
6 543	25.32	5.01	65 845	5.05	6.0	13 663	12 451	16.45
4 209	20.28	5.01	34 106	3.25	6.0	18 865	18 577	13.62
9 464	18.60	5.02	577 800	3.70	8.0	35 591	31 362	13.73
4 112	18.36	5.55	63 490	3.30	....	54 556	55 073	9.96
10 337	25.78	5.45	187 728	4.72	6.0	47 899	45 415	16.50
8 449	17.82	5.55	37 609	3.21	8.0	24 750	23 895	11.93
6 568	20.87	4.62	67 188	4.52	3.5	35 877	36 846	13.70
12 863	25.35	5.06	530 259	5.01	8.0	47 803	44 675	18.58
3 243	8.92	4.73	22 933	1.88	2.5	16 273	16 967	18.17
$19 038	25.89	4.94	$11 644 945	5.24	....	$63 794	$61 928	18.84
$8 002	20.99	4.92	$2 677 358	4.26	....	$37 854	$34 379	14.93
4 064	17.05	5.14	30 485	3.31	....	.....	33 333	16.43
3 709	13.39	5.62	53 640	2.38	....	27 181	27 662	10.75
3 969	16.54	5.10	49 626	3.24	6.0	.....	17 480	15.35
8 182	18.26	4.23	139 266	4.31	10.0	18 192	20 564	11.96
3 439	20.92	5.44	12 483	3.85	....	.....	13 774	15.83
$5 180	16.77	4.73	$285 500	3.55	....	$22 321+	$24 832	12.89
Electric City Railways (C 13 omitted).								
$7 825	22.97	5.15	$406 323	4.46	....	$35 441	$34 016	16.63
6 204	19.01	4.80	1 185 493	3.95	....	59 129	62 517	12.04
8 002	20.99	4.92	2 677 358	4.26	....	37 854	34 379	14.93
5 180	16.77	4.73	285 500	3.55	....	22 321	24 832	12.89
$7 213	20.28	4.89	$4 554 674	4.13	....	$42 579	$42 031	14 .11

(*b*) Railways in Rhode Island and Vermont are not treated in this table.

Mr. Davis.

TABLE No. 5.—All Electric Railways

	No. of roads in each State.	Total passengers carried per annum; year ending June 30th, 1886.	Total length of single track operated. (Miles.)	Percentage of operating expenses to gross earnings.	Total passengers carried per mile of single track operated.
Maine	19	14 651 165	182.80	71	79 054
Connecticut	31	39 272 306	387.01	65	101 478
Massachusetts	103	330 889 629	1 590.95	69	207 982
Rhode Island	9	39 979 695	154.88	60	258 133
New Hampshire (1896)	5	6 037 617	51.11	77	109 555
Vermont (1896)	5	1 637 262	27.14	..	.....
Totals and averages	172	432 467 674	2 393.89	67	182 726

TABLE No. 6.—Totals and Averages of Table No. 3. Electric Electric City Rail-

Maine	†11	14 314 166	139.38	73	102 698
Connecticut	†20	38 816 071	345.41	64	112 376
Massachusetts	†60	145 637 278	1 197.94	66	121 572
Rhode Island	....	.....	.....	..	.....
New Hampshire	†5	6 037 617	55.11	77	109 555
Vermont	....	.....	.....	..	.....
Totals and averages	†96	204 805 132	1 737.84	67	117 890

TABLE No. 7.—Totals and

Table No. 3.—Electric Suburban and Interurban Railways (selected from total number in New England)	†62	111 839 855	1 106.45	65	101 079
Table No. 4.*—Electric City Railways (selected from total number in New England)	†34	92 965 277	631.39	69	147 237
Table No. 5.—All Electric Railways in New England (total number in New England making reports to R. R. Coms.)	†172	432 467 674	2 393.89	67	182 726
Table No. 6.*—Electric City, Suburban and Interurban Railways (selected from total number in New England)	†96	204 805 132	1 737.84	67	117 890

† No. of roads taken.

IN NEW ENGLAND (R. R. COM. REPORTS). Mr. Davis.

Total car-miles operated per annum.	PASSENGER EARNINGS.				No. of passengers carried per car-mile operated.	Percentage of dividends paid, year ending June 30, 1898.	Permanent investment per mile of single track owned: year ending June 30, 1898.	Capital stock and debts per mile of single track owned: year ending June 30, 1898.	Total operating expenses per car-mile operated. (Cents.)
	Per mile of single track operated.	Per car-mile operated. (Cents.)	Per passenger per trip. (Cents.)	Total per annum from passengers; year ending June 30, 1898.					
3 273 932	4 720	26,35	5,89	862 885	4,47	.....	29 331	32 263	18,18
10 323 464	5 253	19,45	5,14	2 018 986	3,80	.....	51 177	51 610	12,71
68 206 418	10 632	24,80	5,11	16 545 554	4,85	.....	43 229	44 958	17,11
.....	13 092	.....	5,07	2 027 693	.....	.....	.....	75 698	.....
1 701 393	5 180	16,77	4 73	285 500	3,55	.....	22 321	24 832	12,89
.....	.....	.....	.....	.....	.....	.....	20 362	30 033	.....
83 505 207	9 186	23,60	5,04	21 740 618	4,68	.....	42 717	46 453	16,52

SUBURBAN AND INTERURBAN ELECTRIC RAILWAYS, PLUS TABLE NO. 4, WAYS (*C* 13 OMITTED).

3 513 376	5 225	20,73	5,08	728 324	4,07	.....	31 297	28 614	15,14
10 135 449	5 548	18,91	4,93	1 916 518	3,83	.....	54 850	57 026	12,23
34 790 116	6 160	21,21	5,06	7 379 549	4,18	.....	32 196	31 585	14,18
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1 701 393	5 180	16,77	4,73	285 500	3,55	.....	22 321	24 832	12,80
.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
50 140 334	5 932	20,56	5,03	10 309 891	4,08	.....	36 313	36 189	13,91

AVERAGES, TABLES NOS. 3, 4, 5 AND 6.

27 682 963	5 201	20,79	5,14	5 755 217	4,04	.....	32 738	32 855	13,20
22 457 371	7 213	20,28	4,89	4 554 674	4,13	.....	42 579	42 031	14,11
83 505 207	9 186	23,60	5,04	21 740 618	4,68	.....	42 717	46 453	16,52
50 140 334	5 932	20,56	5,03	10 309 891	4,08	.....	36 313	36 189	13,91

* *C* 13 omitted.

Mr. Davis.

TABLE No. 8.

Item.	N. Y. C. & H. R. R. R.		Electric Railway (Approximate).	
	Cost per mile of single track.	Per-centage.	Cost per mile of single track.	Per-centage.
1. Grading and masonry	$7 749	18.9	$2 050	5.0
2. Bridges, etc.	1 066	2.6	410	1.0
3. Superstructure	11 439	27.9	9 020	22.0
9*a*. Engineering ($\frac{1}{2}$)	554	1.35	410	1.0
A. Rail bonding	.....	.....	410	1.0
B. Overhead electric line	.....	.....	2 460	6.0
C. Paving	.....	.....	5 740	14.0
Total construction	20 808	50.75	20 500	50.0
4. Stations, etc.	5 453	13.3	.....	.....
5. Land and land damages	5 576	13.6	.....	.....
Total land and buildings	11 029	26.9	8 200	20.0
6. Locomotives	2 337	5.7	.....	.....
7. Passenger cars	574	1.4	8 200	20.0
8. Freight cars	5 576	13.6	.....	.....
9*b*. Engineering ($\frac{1}{2}$)	553	1.35	.....	.....
10. Floating equipment	123	0.3	.....	.....
D. Electric feeder line and ground return	.....	.....	4 100	10.0
E. Central power plants	.....	.....	4 100	10.0
Total equipment	9 163	22.35	12 300	30.0

From Table No. 8, we see that items *A*, *B*, 7, *D* and *E* constitute those which would be an increase in the cost of a new electric line under steam-railroad conditions, or to change an existing steam railroad; all other items have already been shown to be the same, although varying, as the table shows, in the case of a street railway. We therefore get an approximate general figure of $20 000 per mile of single track as the increased cost of the electric railroad. This figure should be treated with caution in any individual case, for it might be less or even exceeded by more than 100% under some circumstances, although it is approximately accurate as an average.

In either a new or an old line there is a limiting factor which sometimes precludes the possibility of using electric traction—the transmission of electric power from the station to the motor. Some problems show at once that, owing to the long distance and infrequent trains of great weight, electric transmission of power would be entirely too costly. Just where the line occurs is hard to define, as it depends upon distance, amount of power, frequency of trains, and their weight. A few years ago it was considered that about ten miles' radius was the limit for one power house; this was before the days of "boosters" and multiphase currents. The development in multiphase transmission has led many to believe that this would make it

possible to successfully operate trunk railroad lines by electric motors. The basis for this is the reduction of line cost by the use of high potentials and the utilization of cheap sources of power such as waterfalls; but it must be remembered that the problem is not solely a question of the original cost of power, but rather the concentration of power and its long-distance transmission coupled with a cheap source, *versus* operation by small isolated units—steam locomotives. Mr. Davis.

In the operation of railways, whether electric or steam lines, one requisite is that the speed of trains must be controlled by the operator; the variations of speed must be numerous, and the change from one to another must be sufficiently smooth to cause no discomfort to the passengers or excessive strain on any of the apparatus. The electric railway motor operating by direct current has long been used for this work with satisfactory results, and in its latest forms is sufficient and serviceable, under the rough usage to which it must necessarily be subjected.

With alternating-current motors the regulation of speed is more difficult. Until the recent development in "induction" motors, the so-called "synchronous" motor was the only type worthy of practical consideration. This type derived its name from the fact that of necessity it ran at the same speed as the generator which supplied it with current. Obviously, therefore, the speed could not be varied. Besides this, the "synchronous" motor would not start by itself under load. Hence it was unsuited for operating railway cars. The other form of alternating motor—the "induction" motor—is capable of speed-regulation to a limited extent, and is continually being improved, so that the future may see developments and improvements beyond our most sanguine expectations. Alternating-current "induction" motors have certain characteristics which would be of great advantage in railway work. The most important of these is the absence of the commutator. In other words, there is no electrical connection between the stationary parts of the "induction" motor and the armature or rotating part, and therefore, there is no exposed metal carrying current, and no sparking due to the use of brushes.

The use of multiphase currents has the following advantages:

(1) They allow some control of the speed of motors.
(2) They provide for starting motors when loaded, but not when over-loaded to the same extent as direct-current motors.
(3) They facilitate insulation.
(4) They overcome some of the effects of induction.
(5) They reduce the cost of conductors.

In short, the "induction" motor, like the "synchronous" motor, presents the difficulties of starting under loads as well as speed-control. The two-phase and three-phase "induction" motors have

Mr. Davis. overcome these difficulties to a certain extent, and they offer some improvements over direct-current motors.

(1) They require less attention.
(2) They give less trouble from dirt, dust or water.
(3) They have no commutator.
(4) They have no moving electrical contacts (except in special cases).
(5) There is less danger of burning out armatures.
(6) They require renewal of parts less frequently.

The disadvantages in their use can be summarized thus:

(1) They do not start readily with over-loads.
(2) Their speed cannot be controlled readily.
(3) Their efficiency is lower.

At present the disadvantages outweigh the advantages, in so far as the conditions of railway work are concerned.

It seems, therefore, that the direct application of alternating-current motors to railroad systems is not feasible to-day from a practical standpoint; mainly for the reason that they do not start under heavy over-loads and are not easily varied in speed over wide ranges.

Irrespective of the motors, however, the alternating current can be used indirectly in operating railway lines. The method, in brief, is to carry the alternating current from the power station to a point in close proximity to a section of the track. Here the current is changed from alternating to direct by means of the so-called rotary transformer, and then applied as direct current to the cars using direct-current motors with the usual devices for controlling the speed. This method of using the alternating current introduces an extra transformation with its concomitant loss, and its use is confined to cases where it is desired to transmit power from a distance. Alternating currents are better adapted than direct currents for transmitting power over long distances, because, with the use of the latter, we are now limited to comparatively low pressures, either in the generating plant (1 000 to 3 000 volts) or the motors (500 to 1 000 volts); while with the former we can "convert," and thus utilize, between the generator and the motor, higher pressures than would be safe at either end. In transmitting currents of electricity over long or short distances, there is a necessary loss in transmission, due to the power required to force the current through the wires. This loss is present in all circuits carrying a current of electricity in greater or less amount, depending upon the volume (amperes) of current (squared), and the quality, size and length of the wire. In cases where large currents are to be carried long distances, the loss is so increased that it requires special consideration. The principle involved in all long-

distance transmission lines is to increase the pressure (voltage) to as high a point as practicable. This is done because the same amount of power can thereby be transmitted, at a correspondingly reduced current requiring a smaller wire, at the same or less loss, since the power is represented by the product of the current and the pressure. Hence, by reducing the current, the energy lost in the line is reduced in proportion to the square of the current. By thus increasing the pressure and reducing the current, the original investment in the transmission line can be kept within reason, and the interest account reduced, as well as the loss of energy in the line. Mr. Davis.

The limit to which this process of reducing the loss in transmission can be carried depends upon our ability to insulate wires carrying high pressures. There is no difficulty in using an alternating current running at a potential of 1 000 to 2 000 volts; for these pressures are common in the outside mains of lighting companies supplying alternating current for incandescent lighting in cities and towns. For long-distance transmission, however, the pressure must be raised to at least 10 000. This is now done successfully in a number of cases. The Niagara Falls transmission line is operated at 11 000 volts, and it is the intention to increase this to 22 000 volts, if it has not already been done; while there are installations on the Pacific Coast where higher potentials are used. How soon higher pressures than those now used will become commercially practicable remains to be demonstrated by the development in the use of present electrical insulating materials, or the discovery of new. In order to obtain high pressures, an intermediate step is required between the generator and the line, known as the "step-up" transformer, and another at the other end of the lines, known as the "step-down" transformer. Generators are built at present to deliver not much more than 2 500 volts, and usually run from 1 000 to 2 000 volts. The "step-up" transformers take this voltage and raise it, for transmission, to 10 000 or 20 000 volts. At the other end of the line the voltage is reduced to the desired amount, in a similar manner, by a "step-down" transformer and then applied to the distributing lines, or to the rotary transformer, or to both, as the case may require. High insulation is therefore required from the "step-up" transformer along the transmission line to the "step-down" transformer. The required insulation is secured by special construction in the transformers, which are also filled with oil, and by insulators of special design attached to cross-arms and poles adapted for the purpose.

We therefore see:

(1) That the alternating-current motor of to-day cannot be applied directly to operating railroad trains, either on the axles of the cars, or at the head of the train as a locomotive, because of the difficulties in starting under load and in controlling the necessary variations in speed.

Mr. Davis. (2) That the alternating current can be applied indirectly to operating railways, because of the facility with which we can transform it to high potentials for long-distance transmission.

Whether it will pay to utilize a "cheap source" such as a waterfall, or concentrate the power into one station on the line of railroad, is, again, the question of balancing the interest account on the added investment for long-distance transmission lines, etc., by reduced operating expenses or increased receipts.

The reduction of first cost to a minimum consistent with work best suited for the purpose is of great importance in the construction of any line. No increase is justifiable, unless it will prove a profitable investment in itself; no decrease, however, is justifiable if the larger expenditure promises larger profit. Where expenditures can be decreased, without materially lessening immediate traffic or adding to operating expenses, such decrease should be made. The importance of this is seen when we remember that the fixed charges (interest on bonds and rentals) are really a part of the cost of manufacturing transportation—to borrow an expression from an eminent authority; they are, in fact, one of the largest items in the total cost of running a railway system; they increase in a somewhat faster ratio than the cost of the line, and are the same every year, whether business is good or bad; and we therefore emphasize the importance of reducing the first cost to the lowest amount consistent with true economy, and of increasing the traffic to its maximum, so that the fixed charges may be a smaller percentage of the gross receipts. It must be remembered, however, that the gross receipts are often enormously increased by extraordinary expenditures, in which case large investments may be warranted, such as terminals at or near large centers of population, shortening of line for moral effect and saving of time in competition, adopting new methods as an inducement to increased use, etc. An idea of the weight which should be given to this part of the subject will be had by the comparison in Table No. 9.

TABLE No. 9.

	Steam Railroads (U. S.)			Massachusetts Electric Railways.
	Average.	High.	Low.	Average.
Percentage of operating expenses...	65	70	55	69
Percentage of net receipts.....	35	45	30	31
Percentage of fixed charges.....	25	45	14	15
Percentage of profit.....	10	16	4	16

By examining Table No. 9, which is not supposed to show highest and lowest figures, but averages under ordinary conditions, we see the

importance of the item of fixed charges, and the importance, therefore, of reducing the first cost. Mr. Davis.

The importance of a reduction in first cost, consistent with true economy, is also clearly brought out by:

Fig. 2. Curve 6.—Showing rapid increase in capitalization per mile of track of railways (40% of which is bonded indebtedness, approximately).

Fig. 8. Curve 2-6.—Showing predicted increase in Curve 6 of Fig. 2.

Fig. 3. Curve 4.—Showing greater increase in bonded debt as compared with increase of mileage of railroads.

Fig. 9. Curve 3-4.—Showing predicted increase in Curve 4 of Fig. 3.

Table No. 10. Capitalization of Steam Railroads and Street Railways.—Curve 4. Total Bonded Debt 332% increase 1870–1890, as against 213% increase in mileage of the U. S.; and 240% increase in New York, New Jersey, Pennsylvania, Delaware and Maryland, against 84% increase in mileage; and 437% increase in Massachusetts against 205% increase of mileage. The increase of 500% in bonded debt of street railways of Massachusetts in the same period, against 544% increase of mileage, does not disprove the statement, for this apparent decrease is due to the phenomenal growth of street-railway mileage.

2. Total Expenses.

As first cost is against electric traction we must look to reduced operating expenses or increased receipts as justification for its use. Let us examine the former. Again refer to our tabulated divisions of "total expenses," etc.

We have adopted the title "total expenses" in a literal sense, including all expenses of every kind, so that the difference between "total expenses" and "gross receipts," if on the right side, would give the amount which can be considered the actual net earnings of the line. "Operating expenses," which are generally treated as the usual and ordinary expenses of the system, include "maintenance and renewal of way and works," "train expenses," "station, terminal, taxes and general expenses," and "accidents, loss and damages," but not "fixed charges," although the latter form a large percentage of the actual total expenses, and have become a necessity in our industrial and financial development. The operating expenses include many items which do not vary with the amount of business or the character of the alignment—such as salaries of leading officials; maintenance of way, works and parts of the system, against that deterioration which

TABLE No. 10.—(*Continued*).

	Capitalization of Steam Railroads and Street Railways:						Fig. 2.		Fig. 3.	
1	Total miles of track	+	213		84		544		205	
2	Annual increase in mileage	±		11	135		446		4	
3	Total capital stock	+	213		104		215		124	
4	" bonded debt	+	332		240		500		437	
5	" capitalization	+	266		155		248		200	
6	" " per mile of track	+	31		52		20		47	
	Earnings of Steam Railroads and Street Railways:						Fig. 4.		Fig. 5.	
1	Gross traffic earnings	+	167		70		400		150	
2	Total interest on bonded debt	+	280		162		715		235	
3	" dividends on capital stock	+	48		16		405		140	
4	Gross traffic earnings per mile of track	+	10		4			**4**	25	
5	Percentage of expenses to gross earnings	+	9		3			**12**		**4**
6	" " interest paid on bonded debt	−	**1***			14		25		15
7	" " dividends " capital stock	−		52		55		10	**6**	
8	Gross earnings per revenue train-mile	−		33				15		10
9	" expenses " " "	−		15				26		23
10	Percentage of passenger earnings to gross earnings	+	8		4				1	
11	" " freight " "	−		6		7				3
12	" " interest and dividends paid on capital stock and bonded debt	−		25					**4**	
	Passenger Traffic of Steam Railroads and Street Railways:						Fig. 6.		Fig. 7.	
1	Passenger train-miles	+	750		200		364		204	
2	Total passengers carred	+	766		900		507		308	
3	Passengers carried one mile	+	443		300				202	
4	Gross passenger earnings	+	152		71		400		120	
5	Receipts per passenger per mile carried	−		33		17				30
6	" " " carried	−		35		37		20		28
7	" " " train-mile	−		32		19		15		22
8	" " mile of track	+	30		3			**4**	55	
9	Number of passengers carried per mile of track	+	14		128			**24**	60	
10	Miles traveled per passenger	−		14		20				1
11	Passenger rides per capita per annum	+	300		300		204		100	
12	Receipts per capita per annum	+	58		22		146		50	

* Heavy type indicates that the percentage of increase or decrease has not followed the characteristic sign of the curve.

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TABLE No. 10.—Percentage of Increase or Decrease of Steam Railroads and Street Railways from 1870 to 1890.

Curve.	Items.	Characteristic sign of curve.	Steam Railroads. United States.		Steam Railroads. N. Y., N. J., Pa., Del., Md.		Street Railways. Massachusetts.		Steam Railroads. Massachusetts.	
			Per cent. Increase.	Per cent. Decrease.	Per cent. Increase.	Per cent. Decrease.	Per cent. Increase.	Per cent. Decrease.	Per cent. Increase.	Per cent. Decrease.
	True Valuation of Real and Personal Estate:									
1	Total in the United States	+	117							
2	" " New York, New Jersey and Pennsylvania	+	....		44					
3	" " Massachusetts	+					33			
4	Per capita in the United States	+	33							
5	" " New York, New Jersey and Pennsylvania	+			0	0				
6	" " Massachusetts	+						16		
7	Percentage of increase in the United States	−		43						
8	" " New York, New Jersey and Pennsylvania	−				85				
9	" " Massachusetts	−						96		
	Population Statistics:		Fig. 1.		Fig. 1.		Fig. 1.		Fig. 1.	
1	Total miles of track	+	213							
2	Annual increase of mileage	±		11						
3	Total population of the United States	+	63							
4	Percentage of increase of the population of the United States	−		28						
5	Density of the population of the United States	+	60							
6	" " " Massachusetts	+					53			
7	Total urban population of the United States (8000 and up)	+	127							
8	Percentage of urban to total population of the United States	+	45							
9	Total population of New York, New Jersey and Pennsylvania	+			44					
10	" " Massachusetts	+					53			
11	" " New York City	+			60					
12	" " Brooklyn	+			83					
13	" " Philadelphia	+			55					
14	" " served by N. Y. & P. R. T. R. R., including New York, Philadelphia, Brooklyn, Newark, Jersey City, Camden, Hoboken, Elizabeth, Long Island City, and Richmond County	+			71					

Mr. Davis. is produced by time only, irrespective of the amount of business handled or work done; salaries of agents of all kinds, many of whom have to be employed whether business is large or small, etc. All of these might be called the "fixed operating expenses," and, as they often amount to nearly half of the total operating expenses, it will prove instructive and remunerative to keep railroad accounts so as to show this. To reduce the total expenses to the minimum consistent with true economy is as important as to reduce the first cost to the lowest amount, bearing the same axiom in mind.

We have two cases to consider and compare with steam-railroad operation: First, central steam-power plants located along the proposed line; second, water-power located at some distance from the line. The following applies to both cases, unless otherwise indicated:

"Repairs of earthwork to subgrade" would be unaffected by the system used. Repairs of track, fences, cattle-guards, crossings, etc., renewals of ties and ballast, are substantially independent of the volume of traffic; ties wear independently of the tonnage, for the so-called "cutting" of ties is due primarily to the rotting away under the rail foot, accelerated somewhat, but almost inappreciably, by the decayed material cut away by passing loads. The total cost of track labor is affected only by a very small percentage of the whole by any change in the amount of traffic, provided the standard of maintenance is not increased; rail-renewals, track-watchman, and part of the labor of lining, surfacing, etc., directly applicable to the track itself, are the only items which vary directly with the number of trains. One-half to three-fourths of the wear of steel rails is caused by the locomotive, and is due more to the great concentration of load on the driving wheels, which often nearly approaches the ultimate crushing resistance of the rail, than to the "hammer-blow" of the reciprocating parts. Maintenance of crossings would be unaffected by the system used. For the reasons enumerated above, items 2, 3, 7, 8 and 9 would be unaffected in cost of maintenance, whichever system were used. While some might claim that the renewal of rails would be less frequent on an electric system, it must be remembered that an electric locomotive or a motor car at the head of a train must have concentrated on the wheels the same weight for the same speeds and loads, the only advantage being the removal of the hammer-blow of reciprocating parts. Should motors be placed on every axle, there would be a saving. In steam-railroad practice this item amounts to about 4% of the total cost per train-mile (see Table No. 11), so that, should there be a saving of 25%, it would equal about 1% of the total cost per train-mile.

Maintenance of bridges and trestles should be slightly less with the use of electric motors, because of less vibratory effects; maintenance of masonry, buildings and signals would not vary in cost with change

of system. We find under "maintenance and renewal of way and works," that there are no items exclusively applicable to steam-locomotive practice; but there are six which apply to electric traction only. This arises naturally from the fact that these items occur under "first cost," and must be maintained. They are: maintenance of—(4) ground-return circuit, (5) overhead-trolley line or third rail, (6) electric feeder lines; and renewals to—(10) poles or insulation of third rail, (11) trolley or third rail and ground return, and (12) feeders. The maintenance and renewals of these constitute an increase in the total expenses of an electric road over those of a road operated by steam. Mr. Davis.

The additional expense of these items, on the basis of all Massachusetts street railways, is shown in Table No. 11. This is, of course, not strictly accurate, as they do not operate under like conditions, but the figures are indicative, so that, with proper consideration of this difference, they can be used:

(*e*) 4.	Repairs of ground-return circuit	0.50	per cent.
(*e*) 5.	" " overhead trolley	1.12	"
(*e*) 6.	" " feeder lines	0.60	"
(*e*) 10.	Renewal of poles for overhead trolley	0.30	"
(*e*) 11.	" " overhead trolley	0 20	"
(*e*) 12.	" " feeders	0.05	"
	Total	2.77	"

The probability is that these percentages are too low for the heavy powers and high speeds of an electric road operating under steam-railroad conditions.

The next division is that of (*b*) "Train Expenses." The relative consumption of coal by steam locomotives and electric motor cars or locomotives, while of importance, is not so large a percentage of the operating expenses as is often imagined. As a rule, the attempt to save in fuel account has had more attention on electric roads than was warranted, and has been made at the sacrifice of other and more important items.

Omitting results obtained from compound locomotives, the records of which are not yet as extensive as those for single expansion, but which should show an average saving of approximately 20%, it has been found that in steam-locomotive practice, of the character we are discussing, the coal consumption per passenger-car-mile (exclusive of engine) averages about 6.5 lbs. for cars weighing 20–25 tons on way-trains, or 30–35 tons on through trains, and that about an average of 1 lb. is added per car-mile for each 6 tons' added weight of car. The average coal consumption of a passenger engine (say 40–45 tons) is about 20 lbs. per locomotive-mile. If we assume six cars to a train, as per our example, we have 3.3 lbs. per car-mile for the engine and 9.8 lbs. per car-mile for the whole train, including engine. This conforms to present average practice of steam railroads.

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TABLE No. 11—Comparative Operating Expenses of Steam

Ordinary division of items in total expenses.	All Massachusetts Street Railways (Railroad Commission Reports, 1899).		
	Total expenditures. (In nearest $1 000.)	Percentage of total operating expenses.	Operating expenses per car-mile.
	(1)	(2)	(3) Per cent.	(4) Cents.
(a) *Maintenance and Renewal of Way and Works:*				
1. Repairs of earthwork to subgrade	$1 163 000 includes (1), (3), (4), (7), (8), (9), (13).	† 9.3	1.6
2. Repairs of track				
3. Repairs of fences, crossings, etc.				
(e) 4. Repairs of ground-return circuit				
(e) 5. Repairs of electric line (third rail or overhead trolley)	286 000 includes (6), (10), (11), (12), (15).	† 2.3	0.4
(e) 6. Repairs of feeder lines				
7. Renewal of rails				
8. " ties				
9. " ballast				
(e) 10. " third rail supports or poles for overhead trolley				
(e) 11. Renewal of third rail or overhead trolley				
(e) 12. Renewal of feeders				
13. Repairs of masonry, bridges, trestles, etc.				
14. Repairs of buildings	91 000	0.7	0.1
15. " signals				
(e) 16. Removal of snow and ice	280 000	2.2	0.4
Totals	$1 820 000	14.5	2.5
(b) *Train Expenses:*				
(s) 1. Fuel for locomotives				
(e) 2. " central power stations	1 268 000 includes (3), (4), (14), (15).	† 10.2	1.7
3. Water supply				
4. Oil and waste				
(s) 5. Repairs of locomotives and tenders				
(e) 6. Repairs of electric motors (car equipment)	439 000	3.4	0.6
7. Repairs of passenger cars	727 000	5.8	1.0
(s) 8. " freight cars				
(s) 9. Use of foreign passenger and freight cars				
(s) 10. Locomotive services (wages)				
(e) 11. Motor car services (wages similar to 10 and 12 for steam railroads)	4 893 000 includes (12).	39.1	7.0
(s) 12. Passenger train services (wages)				
(s) 13. Freight train services (wages)				
(e) 14. Repairs of central power stations				
(e) 15. Central power station services (wages)				
Totals	$7 327 000	58.5	10.3

† Estimated.

RAILROADS AND ELECTRIC RAILWAYS, WITH ITEMS IN DETAIL. Mr. Davis.

Fair average percentage in steam railroad practice.	Percentage of operating expenses on Massachusetts Electric Street Railways by distributing Column 3 according to Column 5.	Probable average percentage of operating expenses all Massachusetts Electric Street Railways estimated from Columns 3, 5 and 6.	Proportion of percentage of operating expenses of steam railroads, Column 7, estimated for proposed railroad from Philadelphia to New York (N. Y. & P. R. R. R.), to operate in 30 to 35 minutes without intermediate stops.	Probable comparative increase or decrease in cost of operation by a change of motive power from steam locomotives to electric motors.		Operating expenses per car-mile Metropolitan Street Railway Company, New York City, 1899.
				Per cent. Increase.	Per cent. Decrease.	
(5) Per cent.	(6) Per cent.	(7) Per cent.	(8) Per cent.	(9)	(10)	(11) Cents.
1.5	0.6970	0.20	1.5	Unaffected.		
8.2	3.8130	4.50‡	5.0	"		0.20
0.3	0.1395	0.02	0.2	"		
	0.0005	0.50	—	0.50		
		1.12†	—	1.12		0.29
		0.60†	—	0.60		
4.0	1.8600	2.50	2.0		1.50	
3.0	1.3950	1.24	0.5	Unaffected.		
1.0	0.4650	0.30	1.0	"		
		0.30†	—	0.30		
		0.20†	—	0.20		
		0.05†	—	0.05		
2.0	0.9300	0.04	1.5		0.05	
2.0		0.70	2.0	Unaffected.		0.06
1.0		0.03†	1.3	"		
		2.20				0.13
23.0		14.50		2.77	1.55	0.68
8.0						
		5.40	—		2.60	0.98
0.6		0.30	1.0		0.30	0.14
1.0		0.50	1.0		0.50	0.06†
6.0						
		3.40	—	0.20		0.475
3.0		5.80	3.0	Unaffected.		1.285
6.0				"		
2.0				"		
7.4						
		39.10	4.0	Unaffected.		6.46
3.0				"		
5.0				"		
		2.00	—	2.00		0.06
		2.00	—	2.00		0.33
42.0		58.50		4.20	3.40	9.79

‡ Includes paving.

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TABLE No. 11 (*Concluded*).—Comparative Operating Expenses of

Ordinary division of items in total expenses.	All Massachusetts Street Railways (Railroad Commission Reports, 1899). Total expenditures. (In nearest $1 000.)	Percentage of total operating expenses.	Operating expenses per car-mile.
(1)	(2)	(3) Per cent.	(4) Cents.
(c) *Station, Terminal, Taxes and General Expenses:*			
1. Agents and station services (wages)			
2. Station supplies			
3. Telegraph			
4. Taxes	855 000	6.8	1.1
5. General officers and clerks	625 000 includes (2), (8).	5.0	0.6
6. Legal	119 000	0.9	0.1
7. Insurance	310 000	2.5	0.4
8. Stationery and printing			0.2
9. Agencies and advertising			
10. Contingent and miscellaneous, not included elsewhere)	827 000 includes (1), (3), (9).	6.6	1.1
Totals.	$2 736 000	21.8	3.5
(d) *Accidents, Loss and Damages:*			
(8) 1. To freight			
2. " passengers	645 000 includes (3), (4).	5.2	0.8
3. " property owned			
4. " " not owned			
Totals	$645 000	5.2	0.8
(e) *Interest Account* (discounts and temporary loans)	523 000		Total 17.1 cents per car-mile.
(f) *Fixed Charges:*			
1. Interest on bonded debt	$1 384 000		
2. Rentals	1 280 000		
Total	$2 664 000		
(g) *Dividends*	2 076 000		

STEAM RAILROADS AND ELECTRIC RAILWAYS, WITH ITEMS IN DETAIL. Mr. Davis.

Fair average percentage in steam railroad practice.	Percentage of operating expenses on Massachusetts Electric Street Railways by distributing Column 3 according to Column 5.	Probable average percentage of operating expenses all Massachusetts Electric Street Railways estimated from Columns 3, 5 and 6.	Proportion of percentage of operating expenses of steam railroads, Column 5, estimated for proposed railroad from Philadelphia to New York, N.Y. & P.R.R., to operate in 30 to 35 minutes without intermediate stops.	Probable comparative increase or decrease in cost of operation by a change of motive power from steam locomotives to electric motors.		Operating expenses per car-mile Metropolitan Street Railway Company, New York City, 1899.
				Per cent. Increase.	Per cent. Decrease.	
(5) Per cent.	(6) Per cent.	(7) Per cent.	(8) Per cent.	(9)	(10)	(11) Cents.
13.0	3.81	0.09	3.0	Unaffected.		.....
3.0	2.08	0.90	0.5	"		.....
1.0	0.29	.....	1.0	"		.....
3.5	.....	6.80	3.5	"		.....
3.5	2.44	3.80	2.0	"		0.19
0.5	.....	0.90	0.5	"		....
0.3	.....	2.50	0.3	"		.....
0.7	0.48	0.30	0.2	"		.....
2.0	0.59	0.01	0.5	"		.....
6.5	1.91	6.50	6.5	"		0.76
34.0	.....	21 80	.....	.....	.....	0.95
0.3	.....	.....	.....	Unaffected.		.....
0.4	2.97	5.09	0.4	"		0.53
0.2	1.49	0.19	0.2	"		Included in (2).
0.1	0.74	0.01	0.1	"		Included in (2).
1.0	.....	5.20	.....	.....	.....	0.53
.....	.....	.....	.....	Unaffected.		Total 11.95 cents per car-mile.
.....	.....	.....	.....	Materially affected, estimated from 5 to 15% or more increase.		.....
.....	.....	.....	.....	Unaffected.		.....
.....	.....	.....	.....	Total percentage of increase 7.02% to 17.02%.		.....
.....	.....	.....	.....			....

Mr. Davis. Table No. 12 shows the average amount of dry steam used per horse-power-hour by various types of stationary engines under different conditions. It is compiled from a large number of actual tests.

TABLE No. 12.—Steam Consumption in Pounds per Horse-Power-Hour.

Type of engine.	Steady Load.		Variable Load 50 to 125 Per Cent.		Extreme Variations—Railway Work, Etc., 0 to 150 Per Cent.	
	Non-con.	Con.	Non-con.	Con.	Non-con.	Con.
High-speed, simple.....	32	28	34	30	36	31
High-speed, compound	23	18	25	21	27	22½
Slow-speed, simple..	25	21	28	23	31½	26½
Slow-speed, compound.....	20	15	22½	18	26	23
High-speed, triple expansion.	17½	13	20	16	.....	.....
Slow-speed, triple expansion.	14½	12½	17	15	.....	.....

Tables Nos. 13, 14 and 15 show the coal consumption per car-mile on electric railways, and were compiled from information obtained in 1894–95. Over 600 railways were communicated with, from which 58 have been selected as giving the most accurate results.

Table No. 13 gives the road by index number, the State where located, the car-miles, kind of boilers, engines and coal, the field from which coal was obtained, the long tons of coal used in the given fiscal year, and the pounds of coal consumed per car-mile.

Table No. 14 gives the average, highest and lowest pounds of coal per car-mile, grouping the roads by territorial sub-divisions and kinds of coal, with the number from which the averages have been calculated.

Table No. 15 gives the average, highest and lowest pounds of coal consumed per car-mile for different classes of engine plants, with the number of roads from which the averages have been calculated.

While the ton-mile should properly be the basis, yet the difficulty of determining the number of ton-miles has led to the adoption of the car-mile as the best practical unit of comparison.

Table No. 12 shows a saving of 36% between the high-speed engine plant running non-condensing at 36 lbs. of steam, and the Corliss compound-condensing at 23 lbs. Table No. 15 shows that the high-speed engine plant averages about 15 lbs. of coal per car-mile, while the Corliss compound-condensing averages about 8 lbs.—a saving of 46%, sufficiently close to the 36% given above to indicate that the coal tables give fairly accurate averages. By examining the coal tables we

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are led to the conclusion that the efficiency of each system is approximately the same, for we find, in general, that, where the number of car-miles is high, the consumption of coal per car-mile is low. In the North Central and Western States, where coal is not of as good quality as in the Eastern States, the consumption is higher (assuming, as we safely can, for our purpose, that the economy of various types of boilers is practically the same); with roads using hard coal there is an apparent higher average in consumption of coal, no doubt due to lower car-mileage and fewer cases; and as more economical apparatus is used in engine plants, the coal consumption decreases. It would seem, therefore, that we are correct in assuming that the total efficiency of each system is approximately the same, though varying in each part that goes to make up the whole. The variation from 25.9 maximum to 5 lbs. minimum shows the average approximate value of this information.

The amount of coal consumed per car-mile is dependent upon so many conditions, often unknown, that this amount, in any given case, cannot be used to predict what the consumption will be on another road; hence the importance of taking a large number of cases upon which to base average results, which may then be considered fairly reliable for the determination of possible results in any new case. It will readily be seen that the amount of coal consumed per car-mile depends upon the tons moved, the height through which each ton is moved, the speed, and the efficiency of the entire system. The first three are almost impossible to obtain on any large number of roads from the ordinary records kept, and can usually be secured only by specially arranged tests. The efficiency of the system depends upon that of each part, such as the boiler plant, engines, condensers and piping, generators, overhead line, ground-return, track, and equipment. High economy in all but one part of the system might cause the pounds of coal per car-mile to be high, and it is therefore impossible, without special test, to determine accurately the cause of low efficiency in any given case. By taking a large number of cases, and eliminating those which are evidently far from the average, we get approximate information indicative of what we may expect.

From Tables Nos. 13, 14 and 15 the average coal consumption per car-mile, under the best conditions of electric street-railway practice, is found to be about 8 lbs. The average weight of these cars, loaded, cannot exceed 8 to 10 tons, so that a six-car train would consume 48 lbs. per train-mile for a maximum weight of 60 tons and average speed certainly not exceeding 10 miles per hour; while with a steam train the consumption would be 58.8 lbs. per train-mile for a maximum weight of 200 tons and average speed of 20 miles per hour. If we compare with compound locomotives, Fig. 10 is of considerable interest. It indicates even a greater saving than 20% which increases as

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TABLE No. 13.

Steam Plant.					Coal.			
Road.	State.	Car-miles.	Boilers.	Engines.	Kind.	Coal field.	Tons used. (2 240 lbs.)	Pounds used per car-mile
North Atlantic States (As per U. S. Census—1890).								
56 *E*	Maine	116 926	V. T.	V.C.	S.	Virginia	508	9.7
42 *E*	Massachusetts.	196 824	R. T.	C.C.C.	S.	Georges Creek, Md..	939	10.7
65 *E*	"	16 794 961	.....	.....	S.	.....	52 878	7.0
189 *E*	"	917 956	W. T.	H.S.C.C.	S.	Georges Creek, Md..	2 201	5.4
74 *E*	"	873 177	.....	.....	S.	.....	2 262	5.8
451 *E*	"	226 123	R. T.	C.C.	S.	.....	504	5.0
308 *E*	Ohio	1 378 518	.....	.....	S.	Pennsylvania	3 189	5.2
802 *E*	Connecticut	146 000	V. T. / R. T.	H.S.C.	S.	.....	1 000	15.3
398 *E*	New York	819 000	V. T.	V.C.C	S.	Pittsburg	2 176	5.9
78 *E*	"	2 162 134	W. T.	V.C.	H.	.....	8 758	9.1
249 *E*	"	637 290	W. T.	H.S.C.C. / C.C.C. \	S.	.....	5 100	17.9
214 *E*	"	199 290	W. T.	.....	S.	.....	1 460	16.4
281 *E*	"	350 000	W. T.	H.S.	S.	Reynoldsville, Pa...	1 200	7.7
181 *E*	"	295 325	R. T.	V.C.C.	S.	Clearfield, Pa.....	682	5.2
80 *E*	"	7 164 258	W. T.	V.C.C.	S.	Reynoldsville, Pa....	23 567	7.4
724 *E*	"	137 970	V. T.	H.S.	H.	.....	1 460	23.7
609 *E*	"	4 558 136	.....	.....	S.	.....	19 043	9.3
842 *E*	"	73 377	R. T.	H.S.C.C.	S.	.....	640	19.5
608 *E*	"	247 482	.....	.....	S.	.....	1 130	10.2
90 *E*	New Jersey	87 923	R. T.	H.S.	S.	Clearfield, Pa.....	387	9.8
558 *E*	Pennsylvania..	149 730	R. T.	H.S.	S.	.....	1 231	18.4
229 *E*	"	600 000	V. T.	V.C.	H.	Hazleton, Pa.....	3 600	13.4
211 *E*	"	514 989	R. T.	C.	H.	Girardville, Pa.....	2 852	12.4
54 *E*	"	400 648	W. T.	H.S.C.	S.	Dawson, Pa.....	1 697	9.4
580 *E*	"	501 400	W. T.	H.S.	H.	Schuylkill Co., Pa..	2 250	10.0
103 *E*	"	833 000	R. T.	C.C.	H.	Liken's Valley, Pa...	3 000	8.1
North Central States (As per U. S. Census—1890).								
648 *E*	Missouri	2 021 938	V. T.	C.	S.	Illinois	16 670	18.5
150 *E*	Ohio	449 680	.....	C.	S.	Jackson Co., Ohio....	2 555	12.7
576 *E*	"	331 000	R. T.	H.S.	S.	Wellston, Ohio.....	1 500	10.1
220 *E*	"	363 040	R. T.	C.	S.	" "	2 300	14.2
222 *E*	Indiana	319 345	V. T.	C.	S.	Illinois and Indiana...	3 140	22.0
107 *E*	Illinois	1 514 802	W. T.	C.C.	S.	.....	9 080	13.3
43 *E*	"	530 000	W. T.	H.S.C. / H.S.C.C. \	S.	Streator, Ill	4 400	18.6
256 *E*	"	425 000	R. T.	C.	S.	" "	3 500	18.4
250 *E*	Wisconsin	150 000	R. T.	H.S.	S.	Duquoin, Ill	1 500	22.4
594 *E*	"	515 000	W. T.	V.C.C. / H.S.C.C. \	S.	Pittsburg, Pa.....	2 190	9.5
170 *E*	Minnesota	1 133 188	W. T.	C.C.C.	S.	.....	5 510	10.9
148 *E*	Iowa	1 549 060	V.W.T.	H.S.C.	S.	Viclo, Ill	11 400	15.5
257 *E*	Missouri	223 130	R. T.	H.S.	S.	Pittsburg, Kan.....	2 090	21.0
648 *E*	"	2 021 938	.....	.....	S.	.....	16 670	18.5
825 *E*	"	395 650	R. T.	C.	S.	Crawford Co., Kan....	2 150	12.2
743 *E*	"	378 995	R. T.	C.	S.	Pittsburg, Kan.....	1 980	11.7
53 *E*	Nebraska	100 000	R. T.	H.S.	S.	Kansas.....	800	17.9
295 *E*	"	1 145 158	V. T. / R. T.	H.S. / H.S.C.	S.	Iowa and Missouri....	9 802	19.2

TABLE No. 13—(*Concluded*). Mr. Davis.

Steam Plant.					Coal			
Road.	State.	Car-miles.	Boilers.	Engines.	Kind.	Coal field.	Tons used. (2 240 lbs.)	Pounds used per car-mile.
South Atlantic States (As per U. S. Census—1890).								
811 *E*	D. of Columbia	534 000	V. T.	H.S.	S.	Cumberland, Md....	1 800	7.5
809 *E*	"	365 000	R. T.	C.	S.	"	1 500	9.2
813 *E*	"	528 500	W. T.	H. S. / H. S. C.	S.	W. Virginia.....	2 030	8.6
128 *E*	Georgia.....	98 550	W. T.	H.S.	S.	Alabama	638	14.5
Western States (As per U. S. Census—1890).								
157 *E*	Colorado.....	3 451 136	R. T.	C. / H.S.	H.	Trinidad, Colo.....	25 000	16.2
901 *E*	"	138 375	R. T.	H.S.	S.	Colorado,.....	1 350	21.8
463 *E*	Utah.....	201 890	R. T.	C.C. / H.S.C.	S.	Rock Spring, Wyo.. / Pleasant Valley, U'h	838	9.3
660 *E*	"	658 416	R. T.	H.S.	S.	Rock Spring, Wyo.. / Pleasant Valley, U'h	3 328	11.3
704 *E*	Washington..	157 832	.....	.....	S.	.....	1 825	25.9
457 *E*	California....	923 694	R. T.	C.C.	S.	.....	2 436	5.9
751 *E*	"	424 529	R. T.	H.S.C.C.	S.	.....	1 250	6.6
Canada.								
796 *E*	Br. Columbia	381 400	R. T.	C.C.C.	S.	British Columbia.....	1 200	7.0
777 *E*	Ontario.....	6 212 045	W. T. / R. T.	C.C.C. / H.S.C.C.	S.	....	18 535	6.7
364 *E*	Quebec	4 888 486	F.	C.C.C.	S.	Cape Breton, N. B....	16 456	7.5

See note under Table No. 15, for explanation of abbreviations.

the speed rises. Average practice gives about 5.5 lbs. of coal per I. H.-P. per hour in single-cylinder steam locomotives, but the character of the service makes it likely to have enormous variations, in any given run. At times this figure is very much exceeded, and also often falls, under favorable conditions, to 4.5 lbs. The conditions which produce such large fluctuations in steam-locomotive service do not exist in the central power station of an electric road. In the latter case we cannot safely figure on less than about $2\frac{1}{2}$ lbs. of coal per I. H.-P. per hour. This may be reduced in exceptional cases, but will more often be exceeded. Apparently, this is a decided saving in favor of the electric road, but it is only apparent. Let us assume the following efficiencies, which are fair averages: Engine, 90%; dynamos, 90%; line, 80%; and railway motors, 75 per cent. In considering these figures it must be remembered that most of the time the larger part of

Mr. Davis. TABLE No. 14.—Averages. Number of Roads and Average Pounds Per Car-Mile.

Locality.	Number.	Pounds Per Car-Mile.		
		Highest.	Lowest.	Average.
All Roads.				
North Atlantic States	26	23.7	5.0	10.65
South " "	4	14.5	7.5	9.95
North Central "	18	22.4	9.5	15.97
Western "	7	25.9	5.9	13.86
Canada	3	7.5	6.7	7.20
Totals and averages	58	....	....	12.46
Roads Using Soft Coal.				
North Atlantic States	20	19.5	5.0	10.01
South " "	4	14.5	7.5	9.95
North Central "	18	22.4	9.5	15.97
Western "	6	25.9	5.9	13.46
Canada	3	7.5	6.7	7.20
Totals and averages	51	....	....	10.38
Roads Using Hard Coal.				
North Atlantic States	6	23.7	8.1	12.78
South " "	..	....	...	.....
North Central "	..	....	...	.....
Western "	1	....	...	16.20
Canada	..	....	...	.....
Totals and averages	7	....	....	13.27

See note under Table No. 15, for explanation of abbreviations.

TABLE No. 15.—Averages (as Per Grouping of Engines).

Engine Plant.	No. of Roads.	Pounds Per Car-Mile.		
		Highest.	Lowest.	Average.
C	9	22.0	9.0	14.5
C. C	7	13.4	5.0	9.2
C. C. C	7	10.9	5.2	8.0
H. S	13	23.7	7.5	15.0
H. S. C	3	16.5	9.4	13.7
H. S. C. C	3	19.5	5.4	10.5

Note.—The following abbreviations are used in Tables Nos. 13, 14 and 15.

F. = Flue.
V. T. = Vertical tubular.
R. T. = Return "
W. T. = Water tube.
V. W. T. = Vertical water tube.
C. = Corliss.
C. C. = " condensing.
C. C. C. = Corliss compound-condensing.
H. S. = High-speed.
H. S. C. = " condensing.
H. S. C. C. = " comp'd-condensing.
S. = Soft coal.
H. = Hard "

the system is running considerably below its rated maximum point of economy; some engineers might place motors at 80%, making the combined efficiency of the entire system about 48.6 per cent. It therefore appears that, for a given average horse-power required at the driving-wheel to move trains, double that amount, or, for safety, even more, must be placed in the central power station, making the coal consumption 5.1 lbs. per I. H.-P. per hour at the motor car, which is but little better than the average of single-expansion steam-locomotive practice, while if compared with compound locomotives (4.4 lbs.) it is higher. It must be remembered, however, that we have been comparing the best electric railway results at low speeds and light weights with average practice in steam locomotives at high speeds and heavy weights; and, while the efficiency of present electric railways is probably less than if they were operated under railroad conditions, it is improbable that this advantage would compensate entirely for the higher speeds and weights which would be moved under such circumstances.

Mr. Davis.

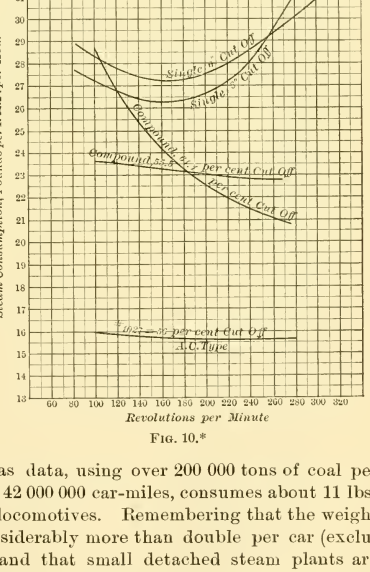

FIG. 10.*

It is interesting to note, as bearing on this subject and as additional indication of the correctness of our position, that a steam road, from which the writer has data, using over 200 000 tons of coal per annum and operating over 42 000 000 car-miles, consumes about 11 lbs. of coal per car-mile in its locomotives. Remembering that the weight moved, in this case, is considerably more than double per car (exclusive of other conditions), and that small detached steam plants are used, running high pressure and single expansion, this is strong corroborative evidence. Turning to Table No. 16 we find that the cost per train-mile on the railroads of the New England States is about 95 cents, or 19 cents per car-mile; this includes freight trains, which, if

* The diagram, Fig. 10, is taken from a paper by Mr. S. M. Vauclain, before the St. Louis Railroad Club, November 11th, 1898.

TABLE No. 16—(*Continued*).

—	23	Average distance traveled per passenger (miles)	17					
+	24	Number of passengers carried one mile (passenger-miles or movement)	1 853 911 708					
+	25	Number of passengers carried per square mile of settled area	1 892	7 500	3.96			
+	26	Passenger trips per capita per annum in New England	23	92	3.96			
+	27	Passenger trips per capita per annum of urban population in towns 8 000 and upward (89 towns)	44	174	3.96			
+	28	Passenger trips per capita per annum of urban population in towns 2 500 and upward (248 towns)	31	123	3.96			
+	29	Passenger trips per capita per annum of urban population in towns 1 000 and upward (819 towns)	25	102	3.96			
+	30	Passenger trips per capita per annum of population served by electric railways, Tables Nos. 28 and 29				43	78	0.55
+	31	Passenger trips per capita per annum of population served by electric railways, Tables Nos. 28 plus 29				74		
+	32	Passenger earnings per capita per annum in New England ($)	7.32	4.64	0.63			
+	33	Passenger earnings per capita per annum of urban population in towns 8 000 and upward ($)	13.88	8.78	0.63			
+	34	Passenger earnings per capita per annum of urban population in towns 2 500 and upward ($)	9.78	6.20	0.63			
+	35	Passenger earnings per capita per annum of urban population in towns 1 000 and upward ($)	8.11	5.14	0.63			
+	36	Passenger earnings per capita per annum of population served by electric railways, Tables Nos. 28 and 29 ($)				2.21	3.86	0.57
+	37	Passenger earnings per capita per annum of population served by electric railways, Tables Nos. 28 plus 29 ($)				3.72		

(*a*) Reports of 1898 are not at hand at this writing, but for the discussion this is not essential, the relative difference being of greater interest than the actual.

(*b*) This number includes all roads making separate reports to the Railroad Commissioners; many of them are owned or leased to one of the large systems operating in the New England States, but for comparison the above figure is correct.

(*c*) While the average number of cars per train for the New England States is not at hand at this time, the Pennsylvania Railroad, from 1874 to 1884, averaged 5.21 cars per passenger train; since that date the number per train has certainly increased.

(*d*) 5 cars per train.

(*e*) Freight and passengers.

TABLE No. 16.—Comparative Statistics of Steam Railroads and Electric Railways.

New England States—Maine, New Hampshire, Vermont, Massachusetts, Rhode Island, Connecticut.

Item number.	Characteristic signs.	Item.	All Steam Railroads. Poor's Manual.	All Street Railways. R. R. Reports and the Writer.	Ratio.	Street Railways in Tables Nos. 3 and 4.		Ratio.
			1897(*a*)	1898.	Street railway returns to steam railroad returns.	Table No. 3. (See Table No. 28.)	Table No. 4. (See Table No. 29.)	Table No. 3 to Table No. 4.
1	—	Number of roads	145(*b*)	172	1.18	62	34	1.82
2	+	Total passengers carried per annum	109 064 367	432 467 674	3.96	111 839 855	92 965 277	1.23
3	+	Total length of single track operated	7 602	2 393.89	0.32	1 106.45	631.39	1.75
4	+	Percentage of operating expenses to gross earnings	74.77%	67.00%	0.89	65.00	69.00	0.94
5	+	Total passengers carried per mile single track operated	14 347	182 726	12.73	101 079	147 237	0.68
6	+	Total car-miles operated per annum	157 429 890(*c*)(*d*)	83 505 207	0.53	27 682 963	22 457 371	1.23
7	+	Total passenger-train-miles operated per annum	31 485 978	83 505 207 +	2.65	.....	.....	....
8	+	Passenger earnings per mile of single track operated ($)	4'527	9 186	2.03	5 201	7 213	0.72
9	+	Passenger earnings per car-mile operated (cents)	21.86(*d*)	23.60	1.08	20.79	20.28	1.02
10	—	Passenger earnings per passenger-train-mile operated (cents)	109.29	23.60+	0.21	.....	.....	....
11	—	Passenger earnings per passenger per trip (cents)	31.55	5.04	0.16	5.14	4.89	1.05
12	+	Total passenger earnings per annum ($)	34 410 597	21 740 618 +	0.63	5 755 217	4 554 674	1.26
13	+	Number of passengers carried per car-mile operated	0.692(*d*)	4.68	6.77	4.04	4.13	0.97
14	—	Number of passengers carried per passenger-train-mile operated	3.46	4.68+	1.35	.....	.....	....
15	—	Percentage of dividends paid	4.96%	.....	.....	.....	.....	....
16	—	Percentage of interest paid	4.56%	.....	.....	.....	.....	....
17	+	Permanent investment per mile of single track operated ($)	.....	42 717	.....	32 738	42 579	0.76
18	+	Capital stock and debts per mile of single track operated ($)	.....	46 453	.....	32 855	42 031	0.78
19	—	Total operating expenses per car-mile operated (cents)	19.00(*d*)	16.52	0.87	13.20	14.11	0.93
20	—	Total operating expenses per passenger-train-mile operated (cents)	95.00(*e*)	16.52+	0.17	.....	.....	....
21	—	Passenger earnings per passenger per mile (cents)	1.86	.....	.....	.....	.....	...
22	—	Number of passenger-miles per passenger-train-mile	58.88	.....	.....	.....	.....	

Mr. Davis. eliminated, would reduce the cost per train-mile and car-mile for passenger trains. Electric railways in the New England States operate at an average of 16.52 cents per car-mile. We also see from Table No. 11 that the average cost of fuel for locomotives is 8% of the total cost per car-mile (passenger and freight), while, for central power stations, it is 5.4 per cent. Combining these figures and allowing for the use of compound locomotives (but making no deduction for freight trains) we have 1.29 cents per car-mile for locomotive fuel and 0.89 cent per car-mile for central power stations. The difference of 31%, apparently in favor of the electric system, in the light of the foregoing data, is due undoubtedly to light weights, low speeds and including freight trains (say 30 cars per train) in the steam-railroad estimate of cost per car-mile, and which would certainly disappear under steam-railroad conditions.

The cost of coal would depend largely upon the amount of power necessary to operate a given road; for the larger the power and more frequent the trains the greater the saving by the use of electric traction. Many central stations, assumed to produce a horse-power at lower cost than by smaller units, really do not, mostly due to the fact that so large a part of such a station is idle many hours of the day; and, although at their maximum point of economy they are far ahead of small units, the question at issue is not "the cost of a horse-power at the station," but "the cost of a horse-power at the rim of the driving wheels." The cost of this last-mentioned horse-power for electric traction is not only made up of station expenses, but also line expenses and electrical equipment expenses, when compared with the cost of the same horse-power produced by a steam locomotive. When these facts are taken into consideration it is invariably found that the controlling factor is the headway of trains; the less the headway, the more chance for electric traction to pay, and *vice versa*, so far as the cost of a horse-power at the rim of the driving wheel is concerned, as affected by the amount of fuel burned. The figures given show this clearly.

The cost of handling coal would be substantially the same with either system, as there would be only one coaling station in either case. The cost of water would undoubtedly be greater in steam-locomotive practice, where it must be supplied at various points along an extended line; but on short suburban roads, to which our discussion is limited, the steam locomotive could make a round trip without requiring water, or would have to be supplied only at each end of the line; and, as the central power station would use much more water on account of larger horse-power and the use of condensing, it is hardly likely that there would be any great difference between the systems. The use of oil and waste on the motor cars should be much less than that which is necessary for the proper care of steam locomotives;

TABLE No. 17. Mr. Davis.

Case 1, Fig. 11.

(*a*) Unlimited amount of water power.

(*b*) Two sources of power, each 100 miles from the line of railway (85 miles long) and dividing the same into ¼, ½ and ¼.

(*c*) Development of these water-powers to cost no more than eleven power stations in Case 2 (this is highly unlikely, but favors Case 1 as against Case 2).

(*d*) Three-phase alternating current of 20 000 volts to be used from source of power to sub-stations, and 1 000 volts direct current from sub-stations to railway motors (this favors Case 1 as against Case 2).

(*e*) Sections, 8.5 miles long (44 892 ft.).

(*f*) Sub-sections, 11 223 ft. long, 30 in all, each with a sub-station rotary-transformer plant.

(*g*) Three tracks, minimum headway on each, three minutes; trains of five cars each, being 8.5 miles apart.

(*h*) Each train will require not more than 10 000 I. H.-P. at the stations, under economical conditions.

(*i*) Each sub-station an economical rating of 20 000 I. H.-P.

(*j*) In either case, feeder lines along the line of railway equal each other (this also favors Case 1 as against Case 2).

(*k*) All other conditions to be the same as assumed in a previous article by the writer.*

Case 2, Fig. 12.

(*a*) Eleven power stations, one for each section and one at each end of the line, 8.5 miles apart, each supplying current to 3 sub-stations with rotary transformers—sub-stations 2.125 miles apart.

(*b*) Cost of the eleven power stations, each 30 000 I. H.-P., economical rating to equal the cost of developing water powers in Case 1 (favoring Case 1 as against Case 2).

(*c*) Three-phase alternating current of 10 000 volts from power stations to sub-stations, and 1 000 volts direct current from sub-stations to railway motors (this favors Case 1 as against Case 2).

(*d*) All other conditions similar to *e*, *f*, *g*, *h*, *i*, *j* and *k*, of Case 1.

* "The Enormous Possibilities of Rapid Electric Travel." *Engineering Magazine*, 1897.

electric motors can have their wearing parts run in oil boxes, which are a protection against dust and dirt, and automatically lubricate the surfaces subjected to friction, but this cannot be done in the case of steam locomotives. This saving in an electric system is, however, counterbalanced by the use of oil and waste in the central power station, so that it is unlikely that there would be much, if any, difference. The figures in Table No. 11 indicate, however, that there is a saving of about 50% in each of these items as compared with steam railroad results.

Thus far we have considered an electric line with power stations located adjacent to the track and with the usual 500-volt direct-current system. If a cheap source of power (waterfall), located at a distance, with the power transmitted to the line, will reduce operating expenses below what they would be by the assumed method, then this saving can be compared to the results of a steam railroad; but, should the reverse be true, the latter comparison is unnecessary. An example will best bring out the facts, and, while the case is hypothetical, a

Mr. Davis. careful examination of the figures will satisfy one that the conclusion is correct, whether or not the estimates in themselves are considered accurate.

Fig. 11 shows the layout of a railroad and feeder lines utilizing a cheap source of power (Case 1), while Fig. 12 shows one for which power is generated by steam plants along the line of the road (Case 2).

Assume the conditions shown in Table No. 17.

From Figs. 11 and 12 it will be seen that the track, third rail, direct-current feeders, sub-stations, etc., will be the same in both

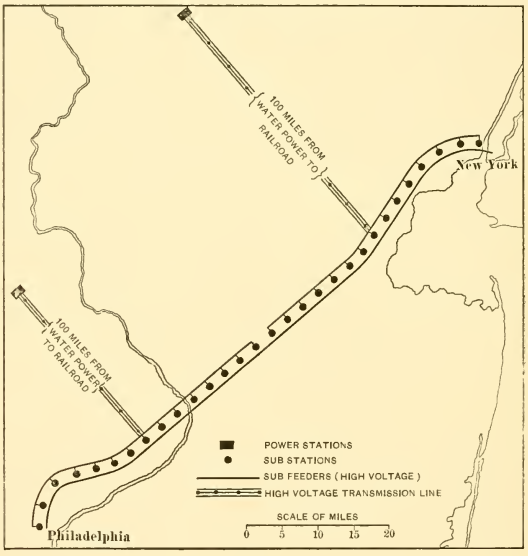

Fig. 11.

cases. The amount of copper in the high-voltage sub-feeders in Fig. 11, which run from the points where the transmission lines strike the road to the sub-stations either way, would be greater than that in the high-voltage lines from the power stations to the sub-stations in Fig. 12. The main point to consider, however, is the high-voltage transmission lines themselves (Fig. 11) from the water-powers to the points where they strike the line of the railroad. Under the above conditions, if the interest on the additional investment for these transmission lines from the "cheap source" of power to the railroad line in Case 1, exceeds the difference between Case 2 and Case 1 in cost of power delivered to

the sub-station supply feeders, then it is unprofitable to utilize the "cheap source" of power. Mr. Davis.

Our calculations, therefore, can be confined to the cost of the two transmission lines in Case 1, each 100 miles long, from the two sources of water-power to the line of railroad; and the difference in the cost of producing power at these waterfalls and the sub-station supply feeders. If the interest on the former exceeds the difference in the latter, then it is unprofitable to use the "cheap source" of power.

In making the calculations, every care has been taken to be fair to the distant source of power, and wherever then is any room for doubt, the "cheap source" has been given the benefit.

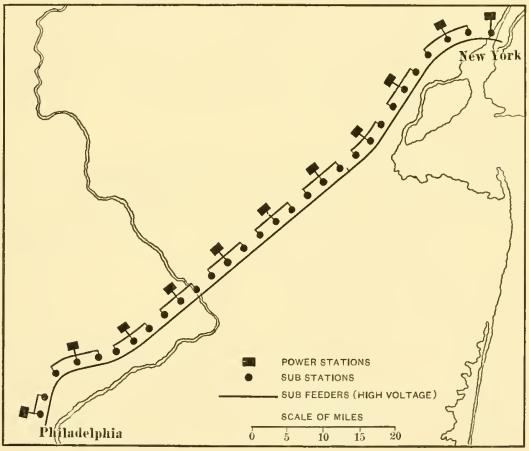

FIG. 12.

The calculation of the pounds of copper necessary is as follows:

$$\text{Circular mils (Diameter of wires squared)} = \frac{\text{Current} \times \text{Distance in feet} \times 20}{\text{Volts lost in line.}}$$

$$\text{Current} = \frac{\text{Watts}}{V} = \frac{\text{Horse-Power} \times 746}{\text{Voltage of transmission.}}$$

$$\text{Pounds of copper} = 0.016 \times \text{circular mils} \times \text{miles.}$$

Therefore,

$$\text{Pounds of copper} = 1\,262\,000\,\frac{\text{Horse-Power} \times \text{miles}^2}{(\text{Voltage of transmission}) \times (\text{Loss in volts}).}$$

For 100 miles transmission at 20 000 volts, and a loss of voltage in the line of 20%, or 4 000 volts the formula becomes:

$$\text{Pounds of copper} = 158 \times \text{Horse-Power.}$$

Mr. Davis. This is correct for direct current. For alternating, the inductive loss would increase the amount of copper required to obtain the same total percentage loss, or, if the same amount of copper were used, would increase the total percentage loss. At the same time there would be a saving due to the three-phase wiring system, which would partly counterbalance the loss due to induction. The actual formula would have to be deduced for each special case of wiring, but, on the whole, the above formula, used directly, is sufficient for our purpose.*

As the horse-power to be transmitted equals 412 500 at 20% loss (330 000 H.-P. delivered at the railroad line), the total amount of copper necessary is 65 175 000 lbs.

We therefore make the comparison shown in Table No. 18.

TABLE No. 18.

	Case 1.	Case 2.
Allowable loss in pressure from water-powers to railway line	20%	.....
Total horse-power to be delivered to railway line at supply feeders of sub-stations	330 000	330 000
Total horse-power at source of power	412 500	330 000
Number of power stations	2	11
Total horse-power at each power station	206 250	30 000
Total distance power is transmitted from each station (miles)	100	0
Pressure at source of power (volts)	20 000	10 000
" " railway line (volts)	16 000	10 000
" lost in line (volts)	4 000	0
Number of sub-stations supplied from each source	15	3
Average length of sub-feeders (miles)	10.625	4.25
Number of sub-stations on each group of sub-feeders	7.5	3
Pounds of copper in transmission line	65 175 000	0

To the total cost of transmission line, amounting to $25 000 000 (in round numbers), should be added the additional cost of developing the two water-powers to 412 500 H.-P. as against 330 000 H.-P. in the eleven steam-power stations along the railroad line. There should also be added the additional cost of feeder lines from the two railroad terminals of the transmission lines to the sub-stations, as against the cost of feeders from the steam-power stations to the sub-stations. While these two items would undoubtedly amount to several million dollars in the case under discussion, we have omitted them, owing to the difficulty of estimating the former without accurate surveys of special localities to be developed; their omission favors Case 1. So far, we have an excess in first cost of $25 000 000 against Case 1 as compared with Case 2.

---

*The correct formula is: Circular mils $= \frac{D \times W}{P \times E^2} \times K$;

where D = distance one way in feet,
W = total watts delivered to consumer,
P = percentage of loss in line of W,
K = 1 690,
and E = potential between conductors at consumers' end of line.

Mr. Davis

Table No. 19 is an estimate of the cost of our proposed transmission line.

TABLE No. 19.

Right-of-way—two strips 100 ft. wide and 100 miles long = about 2 500 acres at $1 000 average	$2 500 000
Engineering, 2% on $15 000 000	300 000
Clearing 600 acres at $100	60 000
Bridges, viaducts, or submarine cables, say (an approximate estimate)	1 500 000
Fencing	200 000
Crossings, say 120 at $1 000	120 000
Signals for guarding and repairing 200 miles at $1 000	200 000
Telegraph, 200 miles at $1 000	200 000
Narrow-gauge repair track and equipment, 200 miles at $5 000	1 000 000
Feeders, 65 175 000 lbs. at 15 cents in place	9 780 000
Pole line, 11 000 poles set in concrete at $100 each	1 100 000
Discount on $25 000 000, 3% 100-year bonds	1 250 000
Interest on bonds to opening of road, say 10 years of construction, averaging $2 500 000 per annum at 3%	4 125 000
Taxes to opening of road, assuming ¼ assessment and 2% rate	1 375 000
Office expenses, salaries, etc., to opening of road, 2% on $15 000 000	300 000
Contingent, etc., not itemized, to opening of road, 5.2%	1 300 000
Total cost	$25 310 000

Table No. 20 is a comparison of the running expenses under the two conditions, based upon the calculations in "The Enormous Possibilities of Rapid Electric Travel," already referred to, the items being given in percentages of the total cost per car-mile run, and being reasonable averages (Table No. 11, Column 8).

TABLE No. 20.*

	Case 1.	Case 2.
Repairs of N. G. track on transmission line right-of-way	1.0%	.....
Repairs of fences, crossings, etc., on transmission line right-of-way	0.1%	.....
Repairs of feeder lines on transmission line right-of-way	6.0%	.....
Renewals of poles and insulation on transmission line right-of-way	1.5%	.....
Renewals of feeder on transmission line right-of-way	1.0%	.....
Repairs of bridges and trestles on transmission line right-of-way	0.3%	.....
Repairs of buildings on transmission line right-of-way	0.1%	.....
Repairs of signals and telegraph on transmission line right-of-way	0.5%	.....
Fuel for power stations	.....	8.0%
Water supply	.....	1.0%
Oil and waste	0.5%	1.0%
Repairs of power stations	2.0%	4.0%
Power station wages	2.0%	4.0%
Average horse-power developed at source of power	412 500	330 000
Total car-miles per annum	43 435 000	43 435 000
Total cost per car-mile	.....	15 cts.
Add for excess power developed by water-powers (25%) say	3.0%	.....

* All other items being the same, the reader is referred to the other articles mentioned in this discussion.

Mr. Davis. From Table No. 20, it is seen that the items added to the cost per car-mile by the use of water-powers located 100 miles away about balance the greater cost of fuel, water, oil and waste, repairs to power stations, and power-station wages, where the stations are located close to the tracks and operated by steam power (each amounts to 18 per cent.). Hence the total expenses would be practically the same for each case. As the transmission lines cost $25 000 000, the interest thereon is the total increase in the expense of utilizing the water-powers instead of steam-power plants. This interest may be figured at 3% ($750 000), 4% ($1 000 000), 5% ($1 250 000), 6% ($1 500 000), or even 10% ($2 500 000), as the case may require, or good judgment dictate. The use, therefore, of these supposed water-powers would decrease the earning capacity instead of increasing it.

Let us, for the moment, assume that our calculations of the operating expenses with the use of water-powers is inaccurate, that in many of the items there is a greater saving, and that those having no counterpart in the other column can be materially reduced.

TABLE No. 21.

Interest.	Car-miles.	Total Interest.	Saving in Cents per Car-mile which must take place to allow the investment.
10%	43 435 000	$2 500 000	5.81
6%	"	1 500 000	3.68
5%	"	1 250 000	2.90
4%	"	1 000 000	2.32
3%	"	750 000	1.74

But 18% (total of percentage items in Table No. 20) of 15 cents (assumed cost per car-mile) equals 2.7 cents; so that, at the five interest rates, we can afford to have our figures wrong by the percentages shown in Table No. 22.

TABLE No. 22.

With interest at 10% — 266% of 18% = (38% of total)
" " " 6% — 133% " " = (24% " ")
" " " 5% — 105% " " = (19% " ")
" " " 4% — 83% " " = (15% " ")
" " " 3% — 64% " " = (11% " ")

It is unlikely that we have made any such error as would be necessary to show that, even with money worth only 3%, it would pay to utilize the water-powers in the case we have discussed. Furthermore, we have favored Case 1 in every way, and more especially by comparing a 20 000-volt system with one of 10 000 volts.

Had we used higher voltages on our transmission line, the copper would have cost:

At 50 000 volts	63 × H.-P. = 10 310 000 lbs.	$1 555 000.
" 100 000 "	25 × H.-P. = 2 600 000 "	390 000.

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By using higher voltages some items of the first cost would be slightly reduced, but, as these constitute more than 60% of the total cost, whereas copper constitutes less than 40%, a reduction in the latter increases the percentage of the former, although reducing the actual amount. Using 100 000 volts, the first cost would not be less than $10 000 000; the interest on this at 3% equals $300 000, and the corresponding saving per car-mile which must take place to allow this investment equals 0.7 cent; therefore 25% of 18% (4.6% of total) can be allowed, even in this case, for errors in our calculation.

The case we have discussed shows conclusively that it would not pay to utilize the water-powers. But, it may be said, this covers only one problem. While this is true, nevertheless this problem is one chosen to enable us to arrive at a general conclusion; for we are dealing with a very large amount of power at a comparatively short distance. The larger the power to be used, the more likely it is that a cheap source will prove profitable; the greater the distance over which the power must be transmitted, the less likely it is that a cheap source will prove profitable.

These calculations have been given to bring out clearly how unlikely it is that, even by the use of alternating long-distance transmission, a trunk line like the Pennsylvania Railroad, for example, will convert its entire system to electric traction. This system, east of Pittsburg, has 2 747.35 miles of railroad, 8 259.36 miles of single track, 1 803 locomotives, 1 765 passenger, baggage and express cars, and 90 527 goods cars. If locomotives were converted at an average horse-power of 300 each, this would make an aggreate of 216 360 H.-P. to be transmitted (allowing 20% in shops and 50% of the total horse-power of the locomotives to be transmitted), while it would have to be taken over many times the distance given in the example. Apply the formula for pounds of copper in transmission line and we see how enormously the weight is increased as the distance increases in the case of the Pennsylvania Railroad. The cost is astonishing when one remembers the claims of an enthusiast who stated: "Where water-power is always available within a few hundred miles" or "for desert railways, where water cannot be obtained," "electric traction is eminently suitable."

Repairs of passenger locomotives and tenders on steam roads average from 4 to 6 cents per mile, with about 1.5 cents to be added for repairs to tools, shops, machinery, etc., properly chargeable to "motive power repairs." Statistics of electric roads are very imperfect and unreliable on matters of this kind, not only because, in the past, such roads have usually been operated by those familiar with horse traction, and having little mechanical knowledge of or inclination toward the precise methods of steam-railroad management and operation, but because of the constant change of apparatus long before the old had given the life which could reasonably be expected of it as a piece of mechanism.

Mr. Davis. Even if we had such data as we might wish from existing electric roads, they would be only indications of what might be expected on a system similar to the one under discussion. From Table No. 11 we find that locomotive and passenger car repairs amount to 9%, while motors, trucks and cars on the electric railroads of Massachusetts cost 9.2%, which again favors the steam railroads; for to this must be added 2% for power-plant repairs, making a total of 11.2 per cent.

When we remember the small horse-power involved, and the light weights and low speeds, it is hardly likely, with the large increase in these necessary under steam-railroad conditions, that the cost of repairs and maintenance of the motive power of an electric road will be any less than that of a steam road. It is to be regretted that more reliable and extensive data are not obtainable. Repairs and maintenance of passenger cars would be the same in either system. The item "use of foreign passenger cars" is the rental paid for the use of cars of other roads while retained, and of course would be unaffected by the use of any particular system. While the operation of a motor car involves far less labor than that of a steam locomotive and its tender, yet the danger that the operator might be incapacitated makes it necessary to have two men at the head of the train, although one man would easily be able to do all the required work. As it takes two men to run the ordinary passenger locomotive, there would be no advantage of one system over the other. It would be a mistake to save wages by utilizing a lower degree of intelligence or skill for motormen and their assistants than is now used for engineers and firemen on steam railroads; the tendency to do this has cost electric street-railway owners many thousands of dollars—more, in fact, than is usually realized. Passenger-train service would not be affected by the use of one system or the other. Central-power-station service is an additional charge against the electric road. Were the operative method changed, train services (wages) would be greatly increased, as shown in Table No. 11.

We now come to (*c*) "Station, Terminal, Taxes and General Expenses," divided into ten subdivisions, none of which is affected by the character of road-bed, rolling stock or mechanical apparatus of any kind, but is dependent upon the character of the business handled, the mode of operation and the volume of traffic; therefore these items are unaffected by the use of any particular motive power. The same statements apply to the division (*d*) "Accident, Loss and Damage," and to (*e*) "Interest Account."

"Fixed charges" includes interest on bonds, and rentals; the actual rate of interest on bonds is not affected by the system used, except in ways beyond the scope of this discussion. We have shown that the first cost of an electric system is considerably greater than that of a steam-locomotive road when operating under the same conditions, and

only in one case will it be less, namely, when the units are small, light and very frequent, when the cost of locomotives would more than equal the natural increases caused by electricity. Mr. Davis.

Therefore the "fixed charges" of an electric system, under steam-railroad conditions will be much higher, and this may vary from 5 to 15% or more as an increase of expenses against the electric road, assuming, of course, that the cost in each case is covered by bonds, which is now usual.

This is no inconsiderable item, as the average fixed charges of our present steam roads are not less than 20% of the gross receipts, and, if this is increased largely by the use of electricity, it is more of a burden than can be overcome by inconsiderable savings in items of operating expenses (if such savings are even possible), such as slightly lower coal consumption, motor car repairs and maintenance, bridge repairs and maintenance, track repairs and maintenance, etc., which may be claimed in favor of electricity by some ardent advocates, irrespective of the intrinsic merits of any given problem.

To recapitulate: We see, by Table No. 11, that the additional charges against electric traction, omitting interest on increased fixed charges, amounts to 6.97%, while the savings, allowing the maximum in each case and all doubtful items, is 4.95%; or 2.02% against the electric system, with the largest of all items not considered.

From what precedes we have shown that the following would be additional charges against the electric road over a steam railroad, and that they would not be inconsiderable, especially the interest charge:

1. Interest on increased cost.
2. Renewal and repairs of transmission line.
3. Repairs of motors and central power plants.
4. Central-power-station services.

And the following the possible, but not necessarily probable, savings:

1. Renewal of rails.
2. Repairs of bridges, trestles, etc.
3. Fuel for central-power plants (unlikely).
4. Water supply (unlikely).
5. Oil and waste (unlikely).

These are items of sufficient importance to necessitate a very careful study of each individual problem before deciding upon a change, or even an original installation.

But some may say that the average of 17 cents per car-mile is high for electric roads, and so it is for a first-class well-managed road, but so is 19 cents for a similar steam road. The Metropolitan Street Railway of New York makes the statement that they operated at 11.95

Mr. Davis. cents per car-mile, while in an article in *Cassier's Magazine* of August, 1899, the statement is made that the Chicago South Side Elevated operated, in the month of December, 1898, at 7.5 cents per car-mile. Both these figures must be taken with a great deal of caution, and for the same reason, namely, the period during which both systems have been in operation has been short, and neither yet knows what its true operating expenses are. Table No. 11 (last column) indicates this. In this table it is seen that the "train expenses" of the Metropolitan Street Railway, most items of which are unaffected by time, are approximately equal to the average of all Massachusetts roads, which have been running several years, while "maintenance and renewal of way and works" are about 25% of the Massachusetts roads. "General expenses" are also about 25%, and accidents about 70 per cent. General expenses were distributed between cable, horse and electric systems; but, assume that these figures are correct, and that an electric road can be operated at 7.5 cents per car-mile, even then they are not approaching the best steam railroad results. The Pennsylvania Railroad Division of that system operated, in 1897, about 5 cars per passenger train and 30 cars per freight train (both figures are probably underestimated); a total train mileage of about 50 000 000 was made, divided into 16 000 000 passenger-train-miles and 34 000 000 freight-train-miles. The cost per train-mile was about 75 cents for the total mileage. Taking 75 cents as the actual cost per train-mile, for either passenger or freight, we would have 15 cents per car-mile as operating expenses for passenger cars and 2.5 cents per car-mile for freight. This method of estimating is obviously wrong, for a freight-train-mile must cost more than a passenger-train-mile, but the former would have to rise above the average to $2.25 per freight-train-mile before the cost per car-mile would increase beyond 7.5 cents. Again, the average weight of an electric car may be taken at 20 000 lbs. (10 tons), so that at 7.5 cents per car-mile the operating expenses would be 0.75 cent per ton-mile, while the Pennsylvania Railroad Division operated at 0.311 cent per ton-mile.

In making a comparison in the use of steam locomotives and electric motors as motive powers, it is seen that only where the units are many, light and frequent, and operated over comparatively short distances, can the use of electricity result in lower first cost or operating expenses; but this involves a change in the method of operation for it to prove desirable. Where units are few, heavy and infrequent, and operated over long distances, the use of steam locomotives will result in lower first cost and operating expenses, and prove more desirable. Though this general statement may be modified by special conditions, such as very high speeds at frequent intervals, it will generally govern in determining changes, or the choice of one or the other, in the case of newly projected roads.

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Without trying to define the exact limit beyond which electricity would be injudicious, we can state that, except for suburban traffic, and interurban traffic between towns a short distance apart, electric traction is to-day higher in first cost and operating expenses; and even when used under such conditions, it may still be so, and success will only come with a corresponding change in operative methods.

The fact that usually the first cost and the total expenses will be greater with electricity than with steam might lead us to discard the thought of using the former under any circumstances, were it not for the question: How will our gross receipts be affected by the use of one or the other system? If by using electric traction we can increase sufficiently the gross receipts per car-mile or train-mile, and per mile of road, we can afford to pay for the additional first cost and greater total expenses. This is the vital question and the real one at issue, although usually not so considered; but, in general, the inducement to adopt electricity instead of steam, or to discard the latter, will be the increased travel which undoubtedly comes, in a greater or less degree, to roads using electric motors.

Freight and miscellaneous receipts cannot be affected by any change of motive power, so our discussion will be confined to the effect on passenger receipts.

3. Gross Receipts.

By examining statistics of steam railroads we find that the volume of travel is influenced by such features as the following: Convenient location of stations with respect to centers of population; length of line; proximity of terminals to the centers of population; number of trains; (the four points just mentioned have an overwhelming effect on short-haul traffic); good road-bed and track neatly kept; handsome commodious stations, with first-class appointments and service; comfortable and luxurious cars; block signals; cleanliness, etc. Profit is proportional to success in obtaining that travel which enables each train to run full, for the cost of a train-mile is not affected by the number of seats taken; and it is this travel which is obtained by a line giving weight to its "location" and "appointments." In freight competition between railroads the rates are ultimately dependent upon the cost from "consignor to consignee," not from station to station, and the railroad eventually pays the additional cost of lighterage, switching, cartage, etc., and the profit thereon, whether by actual difference in rates or loss of business.

The same is true in passenger traffic, as will be seen by free omnibuses, reduction in rates, etc. If you can take the passenger up at his own door and set him down at his place of destination, you have not only suited his convenience (and thus, as we shall see, induced him to travel oftener), but have secured those receipts which otherwise

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TABLE No. 23.—POPULATION

STATES.	Population (United States Census, 1890).	Density (population per square mile).	Total area (square miles).	Area of settlement (square miles).	URBAN POPULATION (CALCULATED FROM U. S. CENSUS 1890.) Towns 8 000 and upward. No. of towns.	Towns 8 000 and upward. Total population.	Towns 2 500 to 8 000. No. of towns.	Towns 2 500 to 8 000. Total population.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Maine	661 086	22.11	29 895	25 729	8	130 346	39	152 677
Connecticut	746 258	154.03	4 845	4 845	17	385 287	49	204 923
Massachusetts	2 238 943	278.48	8 040	8 040	47	1 [illegible]4 931	103	445 018
Rhode Island	345 506	318.44	1 085	1 085	10	272 571	13	55 439
New Hampshire	376 530	41.81	9 005	8 328	5	103 058	22	85 137
Vermont	332 422	36.39	9 135	9 135	2	26 350	22	90 713
Totals and averages	4 700 745	79.79	62 005	57 662	89	2 482 543	248	1 033 907

would have gone to omnibuses, hackmen, and street-car lines. This the steam railroads have failed to do, and it is clear that they cannot altogether do away with these feeders natural to their peculiar modes and conditions of traffic ; but there is little room to doubt that in many cases steam railroads can modify their present methods, for suburban traffic out of large centers of population and for interurban passenger traffic on branch lines between centers of population, by the use of electricity, paying the additional first cost and greater total expenses out of the probable (we might say certain) enormous increase in their gross receipts. This, the writer believes, can be accomplished only by a radical change in the present methods of operation, making them approach, on parts of the system, the present "leave-at-your-door" plan of our street railways, while keeping, on the rest of the system, the present methods of steam railroads with possibly some minor modifications. To accomplish this to advantage, the use of electricity will probably prove advisable, although in some instances a combination of electricity with steam might give the best results. The great increase in gross receipts and in net revenue may, and very often will, decide the question beyond any possible doubt in favor of electricity, always remembering that the adoption of electric traction means a corresponding adaptation of operative methods, and not the mere application of a special form of motive power.

Let us consider some of the figures bearing on the problem.

The Pennsylvania Railroad originally spent about $5 000 000 for its Broad Street terminal; the great St. Louis Bridge and Union Station

STATISTICS (UNITED STATES CENSUS). Mr. Davis.

URBAN POPULATION (CALCULATED FROM U. S. CENSUS, 1890).

Towns 1 000 to 2 500.		Total in Towns 1 000 and upward.		Balance of population not in towns, 1 000 and upward.	Table No. 28.		Table No. 29 (omitting *C* 13).		Table No. 28 plus 29 (omitting *C* 13).	
No. of towns.	Total population.	No. of towns.	Total population.		No. of towns.	Population served.	No. of towns.	Population served.	No. of towns.	Population served.
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
143	209 864	190	492 887	168 199	25	175 676	9	90 219	32	210 887
56	89 894	122	680 104	66 154	52	326 252	57	401 858	77	474 420
107	172 984	257	2 182 933	56 010	144	1 850 837	32	581 668	152	1 961 280
9	14 112	32	342 122	3 384	.....	.....	....	.....	.....	.....
74	105 018	101	293 213	83 317	.....	.....	7	106 673	7	106 673
93	135 831	117	252 894	79 528	.....	.....	....	.....	.....	.....
482	727 703	819	4 244 153	456 592	221	2 352 765	85	1 180 418	268	2 753 260

cost many more millions; while the Market Street terminal of the Philadelphia and Reading Railroad, the Union Station of the Boston and Maine Railroad systems, the Southern Union Station of the New York, New Haven and Hartford Railroad in Boston, and the Grand Central Station in New York are proofs of the millions of dollars spent by railroad companies to locate their terminals at or near the centers of population. These expenditures were incurred almost entirely for the sake of increasing the suburban passenger traffic, although giving a decided stimulus to through and competitive business. In the writer's opinion, our steam railroad companies can afford to double (to use a broad and inaccurate expression) these investments to accomplish what has been suggested above, reaping a handsome return from the increased gross receipts from the passenger traffic thus obtained and encouraged.

Taking the New England States as a basis for comparison—for there we shall find the shortest hauls, the most dense population, the greatest number of trains, and the most accurate statistics—we have from Poor's Manual, Railroad Commissioner's Reports and special calculations by the writer the various tables and figures hereafter discussed.

Effect of Increased Population.—Fig. 1 shows clearly the enormous growth of population and railroad mileage in the United States since 1830, when steam railroads were just coming into existence. Street railways started at about the same time, the first street car operating on the New York and Harlem Railroad tracks, Fourth Avenue, New

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TABLE No. 24.—Growth of Street Railways in 10 Years, 1888 to 1898 (*a*).

Motive Power.	Number of Railways.		Miles of Track.				Number of Cars.				Average Length of Railway (Miles of Track).	
					Percentage to total.				Percentage to total.			
	1888.	1898.	1888.	1898.	1888.	1898.	1888.	1898.	1888.	1898.	1888.	1898.
Electric.....	21	914	86	14 782	1.4	90.2	172	41 402	0.6	87.8	4.09	16.17
Cable..... ..	18	20	217	486	3.6	3.0	2 777	2 920	11.1	6.2	12.06	24.30
Horse	566	112	5 474	683	91.4	4.2	21 736	2 860	86.6	6.0	9.67	6.09
Steam.....	35	27	216	421	3.6	2.6	(*b*)423	(*b*)404	1.7	0.8	6.17	16.59
Totals...	640	1 073	5 993	16 372	.....	.....	25 108	47 586	.....	.....	.....	.....

(*a*) From an article by Mr. William J. Clarke, entitled "Electric Railways in America from a Business Standpoint."—*Cassier's Magazine*, August, 1899.

(*b*) Includes locomotives.

This table is evidently approximate, and was undoubtedly not intended to be exact.

York City, in November, 1832. It would be curious if in the near future (70 years after) some steam railroad should again change its mode of operation so as to approach the methods of street railways. Curves Nos. 7 and 8 show the growth of urban population (according to United States Census those communities which have 8 000 or more inhabitants), while Table No. 23 shows the urban population in Massachusetts down to towns of 1 000 inhabitants.

It is unfortunate that the old horse railways have left so few records, but Fig. 2, showing the increase of mileage in Massachusetts from 1870, gives an indication, while Table No. 24 shows the growth of street railways in ten years, 1888 to 1898. The mileage of street railways (16 000) when compared with that of steam railroads (182 000) shows that the former are confined to thickly populated areas, either within their limits or connecting two or more such areas. Their methods of operation—small, frequent units—adapted itself to such distribution of inhabitants and not to the longer hauls at infrequent intervals shown by the mileage of steam railroads.

The growth of population and industries of all kinds (including transportation) react on each other, favorably or otherwise, depending upon conditions of prosperity. A system like the Pennsylvania Railroad could not have existed in 1830, even though it could have been operated at as low a cost as to-day. It had no suburban short-haul business in its early days.

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The concentration of population into "urban" districts has come through the increased transit facilities of such communities. The horse-roads first enabled a growth which made it necessary for the steam railroads to increase their local facilities, which again reacted on the population to increase it, and then came the cable road to reduce the time spent in getting in and out from the residential sections to the business centers; this, in turn, increased the radius of urban population, and again the steam railroads reached out farther from the center of population; all of which encouraged concentration of inhabitants, and the electric railway once more caused a similar cycle of events.

There is no reason to suppose that these conditions have ceased, and that this action and reaction have stopped. It is this enormous development of the country which has allowed the constant increase of investment on transportation systems, and this in turn has increased the carriage of both passengers and freight. In the growth of transit facilities and methods the "radius of action" of the roads has increased constantly, and as it increased so did the urban population and *vice versa*; but when our cities grow beyond the present "radius of action," our transit systems must take another step. Some communities have already arrived there, and the solution will undoubtedly be the equipment of our steam railroads, running out of and between those near together, with electric traction. This will enable them to change operative methods to approach present street railway practice, thus increasing gross receipts enough to offset interest on increased first cost, and reduce operating expenses per car-mile; for the increase in present investments and traffic have also tended to reduce the cost of transportation per head.

Effect of Increased Capitalization.—Fig. 2 also shows the enormous increase in mileage and investment of street railways in Massachusetts, where the density of population is high and has increased more rapidly than in any other part of the United States, as shown by Curves Nos. 5 and 6, Fig. 1. It is because of this increase, which is still going on, that constant increased investments are warranted on established transit systems. This will induce our steam railroads to handle increased traffic in urban, suburban and interurban areas by small, frequent units, operated by electric motors, and thus also stimulate and again increase a traffic which will only grow so much and then wait to be encouraged.

Curve 6, Fig. 2, shows the tendency toward increased investment per mile; the drop from 1894 to 1896 indicates a greater increase in suburban than urban mileage, costing less per mile and thus reducing the average. Fig. 3 shows how comparatively small was the percentage of increase of steam railroad mileage, but a very distinct increase is shown in investment per mile (Fig. 3 gives totals of roads in Massachusetts, but includes mileage, etc., outside the State).

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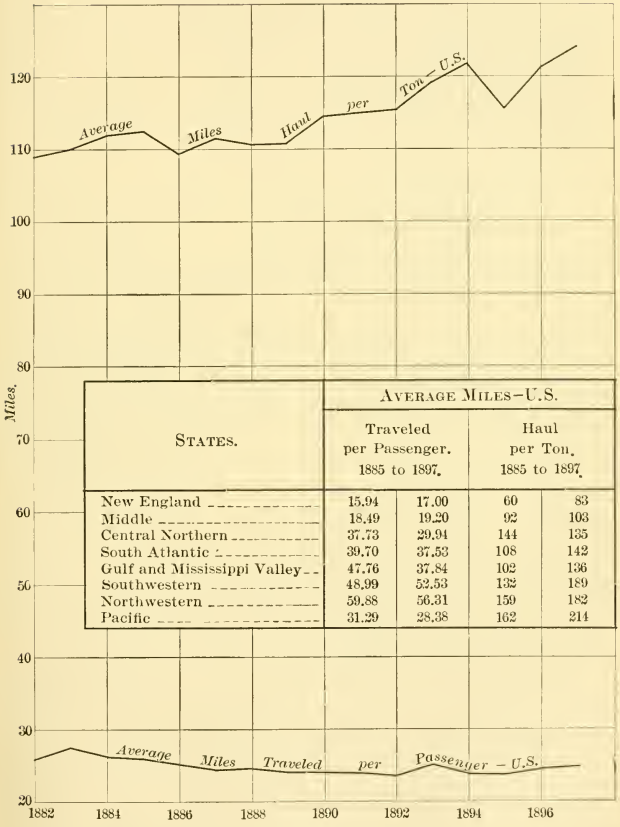
Effect of Increased Earnings.—Figs. 4 and 5 show the earnings of street and steam roads in Massachusetts. Attention is also called to Table No. 16, giving comparative statistics of steam and electric roads, and also to Table No. 10, giving the percentage of increase or decrease in all items (curves) discussed and found in the various figures, between the years 1870 and 1890. We again see the constant increase in gross receipts, especially of the street railways (400% as against 150% for freight and passengers of steam railroads, being 120% for passengers only, Fig. 7, Curve 4); and also the increase in interest and dividends on increased capitalization, some of which is not chargeable to increased mileage (also much larger for the electrics than for the steam).

The gross traffic earnings per mile increased 25% on the steam and decreased 4% on the street roads; the latter is undoubtedly due to the relatively large increase in mileage; this curve, however, has a "characteristic + sign" which will be shown in future growth in the years to come. Curves 8 and 9 in both figures show the characteristic constant reduction in receipts per car-mile or train-mile, due to increased mileage giving better facilities and thereby increasing the traffic at a slightly lower rate, both as to numbers and fares; also reduction in operating expenses per car-mile as the same increases.

Effect of Increased Traffic.—Figs. 6 and 7 (see Table No. 10 also) again bring out the same characteristic comparisons as the previous figures, and, in addition, the following: Curve 10, Fig. 7, indicates that passengers are hauled a shorter distance, on the average, every year. Fig. 13 brings this out somewhat more clearly. In the Eastern States, which are more densely populated, the average distance is approximately constant, but with a tendency to fall. In other words, the proportion of short-haul passengers to total passengers is rising as the population increases and the rides per inhabitant per annum increases.

As the long-distance telephone system develops, its use will grow to such a point that it will no doubt have a decided effect on long-distance passenger traffic, if it has not already. This will increase the proportion of short hauls as compared with long hauls, and the passenger business of our steam railroads (or their partial development into electric railways) in the future, will be more and more largely that of suburban and interurban localities. The increase in total passengers carried was 308% on the steam railroads and 507% on the street railways (see Table No. 16 for totals), while passenger-train-miles increased 204% and 364% (car-miles) respectively.

The receipts per passenger are constantly decreasing as the number of passengers increases, as shown by Curve 6 on each figure. The larger proportional increase in mileage of street railways has reversed the characteristic sign of Curve 4, Fig. 4 (receipts per mile of track), and Curve 9, Fig. 6 (number of passengers carried per mile of track),



States.	Average Miles—U.S.			
	Traveled per Passenger. 1885 to 1897.		Haul per Ton. 1885 to 1897.	
New England	15.94	17.00	60	83
Middle	18.49	19.20	92	103
Central Northern	37.73	29.94	144	135
South Atlantic	39.70	37.53	108	142
Gulf and Mississippi Valley	47.76	37.84	102	136
Southwestern	48.99	52.53	132	189
Northwestern	59.88	56.31	159	182
Pacific	31.29	28.38	162	214

Fig. 13.

Mr. Davis. when compared with Curves 8 and 9, Fig. 7, showing the same items on the steam railroads of Massachusetts. Records do not give the average miles traveled per passenger on street railways. Whether or not there is a temporary increase cannot be determined; but, that in the long run it tends to decrease, there is little room to doubt, although subject to some variation.

Table No. 25 shows the increase in passengers carried per car-mile on street railways and the decrease per train-mile on steam railroads.

TABLE No. 25.—STREET AND STEAM ROADS IN MASSACHUSETTS.

	1870.	1890.
Number of passengers carried per car-mile operated (Street Railways)	4.91	5.51
Number of passengers carried per train-mile operated (Steam Railroads)	4.71	4.30

This shows the increase in train-mileage of steam roads as compared with passengers carried, while the reverse is true of street railways.

Passenger rides per capita per annum and corresponding receipts per capita per annum have increased enormously on street railways when compared with steam railroads. The number of rides increases more rapidly than the receipts per capita; about twice as fast on steam railroads and 35% more on street railways, while the actual increase of the latter has been 204% and the former 100 per cent. In other words, the steam roads have been reducing fares to more nearly approach the low charges of street railways.

These results show most clearly how the low fares (total cost per trip, not per mile) and "leave-at-your-door" service of electric street railways have increased the passenger rides per annum and the number per car-mile so that low fares are profitable. In 1896, the rides per capita per annum were 116 for street railways and 44.5 for steam, while the receipts per capita per annum were $5.95 and $12.80, respectively, for Massachusetts.

Effect of Future Growth.—Figs. 8 and 9 are only indications of what the future has in store for railroads and railway systems in Massachusetts. The predictions have been plotted by taking two points on the street-railway curves corresponding to 1885 and 1895 and extending the straight line to 1910, while the points on steam-railroad curves were 1880 and 1890.

Those curves which when extended give a result contrary to the "characteristic sign" have been thus marked. The number of these shows the indicative nature of the information and its lack of exactness, together with the importance of cross-references to "characteristic signs" and other curves before arriving at a conclusion based on averages in any particular case. These curves might be called "pro-

TABLE No. 26.—Ratios of Comparative Statistics of Steam Railroads and Street Railways. Mr. Davis.

Item number—Table No. 16.	Items.	Ratio of Comparative Statistics, Table No. 16.							
		All electric railways in New England. Table No. 5.	Electric city railways. Table No. 4.	Electric suburban and interurban railways. Table No. 3.	Electric city, suburban and interurban railways. Tables Nos. 3 plus 4.	Electric city railways. Table No. 4.	Electric suburban and interurban railways. Table No. 3.	Steam railroads of New England. Table No. 16.	Street railways of New England. Table No. 16.
1	Number of roads taken	1	0.19	0.36	0.55	1	1.82	1	1.18
3	Length	1	0.26	0.46	0.72	1	1.75	1	0.32
2	Passengers carried	1	0.21	0.25	0.47	1	1.23	1	3.96
12	" earnings	1	0.21	0.26	0.47	1	1.26	1	0.63
6	Car-miles	1	0.26	0.33	0.60	1	1.23	1	0.53 (*a*)
5	Passengers carried per mile	1	0.80	0.55	0.64	1	0.68	1	12.73
13	Passengers carried per car-mile	1	0.88	0.86	0.87	1	0.97	1	6.77
26	Passengers carried per capita per annum	1 (*d*)	0.85 (*d*)	0.47 (*d*)	0.80 (*d*)	1	0.55 (*c*)	1	3.96 (*b*)
8	Passenger earnings per mile	1	0.78	0.56	0.64	1	0.72	1	2.03
9	Passenger earnings per car-mile	1	0.84	0.88	0.87	1	1.02	1	1.08 (*a*)
11	Passenger earnings per passenger	1	0.97	1.02	0.99	1	1.05	1	0.16
32	Passenger earnings per capita per annum	1 (*d*)	0.84 (*d*)	0.47 (*d*)	0.80 (*d*)	1	0.57 (*c*)	.....	0.63 (*b*)
4	Percentage of operating expenses	1	1.03	0.97	1.00	1	0.94	1	0.89
19	Operating expenses per car-mile	1	0.85	0.79	0.84	1	0.93	1	0.87 (*a*)
17	Permanent investment per mile	1	0.99	0.76	0.85	1	0.78	1	.....
18	Capital investment per mile	1	0.90	0.70	0.77	1	0.76	1	.....

(*a*) Estimated 5 cars per train on steam railroads.
(*b*) Population served estimated to be the total in New England.
(*c*) Population served estimated to be that given in Tables Nos. 28 and 29.
(*d*) Population served estimated to be the total in New England for Column 1, and that given in Table No. 29 for Column 2; Table No. 28 for Column 3; and Tables Nos. 28 and 29 for Column 4.

jection" rather than "prediction" curves; but, nevertheless, they indicate strongly the future, with its lower percentage of earnings on capital invested, lower cost of operation, greater percentage of operating expenses and larger gross revenue coming from that enormous future growth in passenger traffic at low fares with electric propulsion.

When the population and real wealth become fixed, then such growth will cease, but only then, and will fluctuate with prosperity, but this condition is far distant for the United States.

Comparison of City and Suburban Railways.—Tables Nos. 3 and 4 have been prepared in an attempt to make the discrimination between

Mr. Davis. roads "in towns" (urban) and "between towns" (interurban). Some of those in either table may more properly belong in the other, but it is believed that the comparative results shown will be borne out in practice.

Suburban roads have been placed in Table No. 3, where their characteristics conformed to the requirements of the table. Tables Nos. 6 and 7 give summaries of Tables Nos. 3 and 4.

From these tables and from Table No. 16 has been calculated Table No. 26, giving ratios of statistics of steam and electric roads. The first five items (1, 3, 2, 12 and 6) show the relative number, length and capacity of roads, for these may affect the other items in the lower part of the table (5, 13, 26, 8, 9, 11, 32, 4, 19, 17 and 18). From this table we have Table No. 27, showing characteristic signs, when comparison is made according to the notation at the top of each column.

TABLE No. 27.

Item.	All Electric Railways in New England to				
	Table No. 4.	Table No. 3.	Table No. 3+4.	Table No. 4 to Table No. 3.	Steam railroads to electric railways, Table No. 16.
5. Passengers carried per mile	—	—	—	—	+
13. " " " car-mile	—	—	—	—	+
26. " " " capita per annum	—	—	—	—	+
8. " earnings " mile	—	+	—	+	+
9. " " " car-mile	—	—	—	—	+
11. " " " passenger	—	—	—	—	+
32. " " " capita per annum	—	—	—	—	+
4. Percentage of operating expenses	+	—	+	—	—
19. Operating expenses per car-mile	—	—	—	—	—
17. Permanent investment per mile	—	—	—	—	..
18. Capital " " "	—	—	—	—	..

The characteristics brought out by these figures are the enormous number of passenger trips per capita per annum, per mile of track and per car-mile, and the greater total number carried by the electric roads when compared with the steam railroads; also the larger earnings per mile of track and per car-mile on the electrics. The receipts per passenger per trip are only 16%, while the receipts per capita per annum are only 63% of the steam railroads. The steam road gets greater receipts per trip, but carries each passenger a longer distance, and has to run a disheartening number of cars or train-miles for the passengers carried. The difference is due to the short trips, high fare per passenger and car-mile, and the "leave-at-your-door"

service of electric roads, while each trip is delivered at an extremely low "total cost" to the passenger. Mr. Davis.

Suburban and interurban railways carry fewer passengers per mile, per car-mile, and per capita per annum than city railways. They earn less per mile of track, more per car-mile and per passenger carried, and less per capita per annum. They operate at a lower percentage of gross receipts and about 7% less per car-mile (Table No. 7), while they cost 25% less. A comparison of such roads with steam railroads is a more accurate one than with city railways, but for obvious reasons it could not be made here.

Effect of Population Served.—Tables Nos. 23, 28 and 29, show the approximate population served by each road in Tables Nos. 3 and 4. In locating any new line, it is essential to know, as nearly as may be, the probable business which it will secure. Upon its amount, the cost of the line completed, and the probable operating expenses, will depend the profit of the undertaking. What other similar roads have done, is the criterion upon which we base all estimates, and the three principal units used are (1) revenue per head of population (number of trips per inhabitant per annum). (2) cost per mile, and (3) operating expenses per car-mile or train-mile. The first is that in which we are now interested.

In determining the gross revenue per head of population, the usual method has been to divide the United States into arbitrary sections, such as the groups of States in Poor's Manual (see Fig. 13), and by dividing the gross receipts by the inhabitants of each division determine the average amount contributed by each individual. Of course, each person does not spend this amount, in fact, some do not contribute anything, while others expend many times the average amount each year. The passenger traffic on any road can be divided into the following general classes:

1. Those who live adjacent to the line and are compelled to ride upon it.
2. Those who are compelled to ride but who can take another line just as conveniently.
3. Those who ride for pleasure.
4. Those who come from a distant place by some other means of conveyance and then ride upon the line for any of the other three reasons.

Those of the first class are by all odds the largest in volume, and as they are the most permanent, increasing or decreasing by known amounts in stated census periods, it would be the best possible basis for our figures of receipts per head of "population served." The difficulty is to get at this "population served." An attempt has been made in these tables, in order to show how receipts would vary according to

Mr. Davis.

TABLE No. 28.—Population Served by Roads in Table

Name of Railway.	Reference No.	Fig. 16. Order according to population served, smallest being No. 1.	Place.	Population served.	Place.	Population served.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Maine.						
Augusta, Hallowell & Gardiner	s 1	19	Augusta	10 527	Hallowell	3 181
Bangor, Orono & Old Town	s 2	32	Bangor	19 103	Veazie	(a)
Biddeford & Saco	s 3	23	Saco	6 075	Biddeford	14 443
Lewiston & Auburn	s 4	41	Lewiston	21 701	Auburn	11 250
Portland & Cape Elizabeth	s 5	44	Portland	36 425	South Portland	(a)
Portsmouth, Kittery & York	s 6	12	Kittery	2 864	York	2 444
Rockland, Thomastown & Camden	s 7	14	Thomaston	3 009	Rockland	8 174
Total and average						
Connecticut.	s					
Danbury & Bethel	s 8	22	Danbury	16 552	Bethel	3 401
Derby	s 9	17	Derby	5 969	Birmingham	2 000*
Fair Haven & Westville	s 10	55	New Haven	81 298	Westville	(a)
Hartford, Manchester & Rockville	s 11	9	Manchester	8 222	S. Manchester	1 500*
Hartford & West Hartford	s 12	54	Hartford	53 230	W. Hartford	1 930
Meriden Electric	s 13	33	Meriden	21 652	Yalesville	(a)
Norwalk St. Ry	s 14	27	Norwalk	17 747	South Norwalk	4 875
Norwalk Tramway	s 15	47	Norwalk	17 747	East Norwalk	(a)
Norwich	s 16	16	Norwich	16 156	Taftville	(a)
Torrington & Winchester	s 17	2	Torrington	6 048	Winsted	(a)
Bristol & Plainville	s 18	6	Bristol	7 382	Plainville	1 993
Central Railway & Electric (New Britain)	s 19	18	New Britain	16 519	Berlin	2 600
Total and average						
Massachusetts.						
Arlington & Winchester	s 20	7	Winchester	4 861	Arlington	5 629
Athol & Orange	s 21	8	Athol	6 319	Orange	4 568
Braintree & Weymouth	s 22	13	Weymouth	10 866	Braintree	4 848
Bridgewater, Whitman & Rockland	s 23	24	Bridgewater	4 249	E. Bridgewater	2 911
Brockton, Bridgewater & Taunton	s 24	53	Brockton	27 294	Bridgewater	4 249
Brockton & East Bridgewater	s 25	40	Brockton	27 294	E. Bridgewater	2 911
Dartmouth & Westport	s 26	59	Fall River	74 398	New Bedford	40 733
Dighton, Somerset & Swansea	s 27	1	Dighton	1 889	Somerset	2 106
Fitchburg & Leominster	s 28	39	Fitchburg	22 037	Leominster	7 269
Gloucester, Essex & Beverly	s 29	45	Beverly	10 821	Wenham	(a)
Greenfield & Turners Falls	s 30	10	Greenfield	5 252	Montague	6 296
Haverhill & Amesbury	s 31	50	Haverhill	27 412	Merrimack	2 633
Haverhill, Georgetown & Danvers	s 32	42	Haverhill	27 412	Bradford	3 720

No. 3. Electric Suburban and Interurban Railways. Mr. Davis.

Place.	Population served.	Place.	Population served.	Place.	Population served.	Population served	Passenger trips per capita per annum of population served.
(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Farmingdale	(a)	Gardiner	5 491			19 699	41
Orono	2 790	Old Town	5 312	Great Works	(a)	28 705	42
Old Orchard	(a)					20 518	11
						32 951	25
Cape Elizabeth	5 459					42 384	25
Portsmouth (N. H.)	9 827					15 135	53
Rockport	(a)	Camden	4 621			16 304	59
						175 676	36
						19 953	41
Ansonia	10 342					18 311	38
Montowese	(a)	Hamden	3 882	Fair Haven	(a)	86 680	56
Manchester Center	(a)	Rockville	1 000*			11 222	53
Farmington	3 179	Unionville	(a)	Plainville	1 993	60 832	6
Wallingford	6 584					28 736	50
Winnipauk	(a)					23 192	32
South Norwalk	4 875	Westport	3 715	Stamford	15 700	46 813	16
Yantic	(a)	Thamesville	(a)	Sunnyside	(a)	18 156	69
Burrville	(a)	Highland Lake	(a)			7 548	65
Forrestville	(a)	Lake Compounce	(a)			10 375	52
Plainville Center	(a)	Newington Center	(a)			19 119	74
						326 252 (b)	43 (c)
						10 490	31
						10 887	55
						15 714	63
Whitman	4 441	Rockland	5 213	Abington	4 260	21 074	25
W. Bridgewater	1 917	Raynham	1 340	Taunton	25 448	60 248	22
W. Bridgewater	1 917					32 122	11
Dartmouth	3 122	Westport	2 599			120 852	4
Swansea	1 456					5 451	142
Lunenburg	1 146					30 452	71
Hamilton	(a)	Essex	1 713	Ipswich	4 439	42 624	27
						11 548	61
Amesbury	9 798	Salisbury	1 316	Newburyport	13 947	55 106	25
Groveland	2 191	Georgetown	2 117			35 440	13

Mr. Davis.

TABLE No. 28 (*Concluded*).—POPULATION SERVED BY ROADS IN

NAME OF RAILWAY.	Reference No.	Fig. 16. Order according to population served, smallest being No. 1.	Place.	Population served.	Place.	Population served.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
MASSACHUSETTS (*Continued*).						
Hingham	*s* 33	3	Hingham	4 564	E. Weymouth	(*a*)
Hoosac Valley (North Adams)	*s* 34	37	North Adams	16 074	Adams	9 213
Interstate Consolidated (R. I.)	*s* 35	46	Pawtucket (R. I.)	27 633	Seekonk	1 317
Leominster & Clinton	*s* 36	20	Leominster	7 269	Lancaster	2 201
Lowell, Lawrence & Haverhill	**s 37**	61	Lowell	77 696	Dracut	1 996
Lowell & Suburban	**s 38**	56	Lowell	77 696	Billerica	2 380
Lynn & Boston	**s 39**	62	Boston	448 477	Beverly	10 821
Marlborough	*s* 40	38	Marlborough	13 805	Hudson	4 670
Millford, Hollister & Framingham	**s 41**	21	Milford	8 780	Hopedale	1 176
Natick & Cochituate	*s* 42	28	Natick	9 118	Wayland	2 060
Newburyport & Amesbury	*s* 43	31	Newburyport	13 947	Amesbury	9 798
Newton	*s* 44	48	Newton	24 379	Waltham	18 707
Newton & Boston	*s* 45	51	Newton	24 379	Watertown	7 073
Norfolk Central (Dedham)	*s* 46	11	Dedham	7 123	Norwood	3 733
Norfolk Suburban (Hyde Park)	*s* 47	43	Dedham	7 123	Hyde Park	10 193
Northampton	*s* 48	25	Northampton	14 990	Easthampton	4 395
North Woburn	*s* 49	36	Woburn	13 499	Winchester	4 861
Providence & Taunton	*s* 50	60	Providence (R. I.)	132 146	Taunton	25 448
Quincy & Boston	*s* 51	49	Quincy	16 723	Weymouth	10 866
Reading & Lowell	*s* 52	4	Reading	4 088	Wilmington	1 213
Rockland & Abington	*s* 53	29	Abington	4 260	Rockland	5 213
Rockport	*s* 54	34	Gloucester	24 651	Rockport	4 087
Southbridge & Sturbridge	*s* 55	5	Southbridge	7 655	Sturbridge	2 074
South Middlesex (Natick)	*s* 56	30	Natick	9 118	Sherborn	1 381
Taunton & Brockton	*s* 57	52	Brockton	27 294	Taunton	25 448
Wakefield & Stoneham	*s* 58	35	Wakefield	6 982	Stoneham	6 155
Warren, Brookfield & Spencer	**s 59**	26	Warren	4 681	W. Warren	(*a*)
Worcester & Blackstone Valley	*s* 60	15	Millbury	4 428	Sutton	3 180
Worcester & Marlborough	*s* 61	58	Worcester	84 655	Shrewsbury	1 449
Worcester & Suburban	*s* 62	57	Worcester	84 655	Millbury	4 428
Total and average						
Grand total and average						

s 2. Also—

Stillwater	(*a*)

s 15. Also—

Roton Point	(*a*)
Rowayton	(*a*)
Darien	2 276
Noroton	(*a*)
Dorlon's Point	(*a*)
	4 276

s 29. Also—

Gloucester	24 651

s 37. Also—

Haverhill	27 412
Groveland	2 191
W. Newbury	1 769
Newburyport	13 947
N. Andover	3 742
	49 061

s 38. Also—

Tewksbury	2 515

s 39. Also—

Lynn	55 727
Malden	23 031
Marblehead	8 202
Melrose	8 519
Peabody	10 158
Revere	5 668
Salem	30 801
Saugus	3 673
Stoneham	6 155
Swampscott	3 198
Wenham	(*a*)
Woburn	13 499
Hamilton	(*a*)
	169 631

TABLE No. 3. ELECTRIC SUBURBAN AND INTERURBAN RAILWAYS.

Mr. Davis.

Place.	Population served.	Place.	Population served.	Place.	Population served.	Population served.	Passenger trips per capita per annum of population served.
(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
N. Weymouth	(*a*)	Hull	2 000*			7 564	119
Williamstown	4 221					29 508	52
Attleborough	7 577	N. Attleborough	6 727	Wrentham	2 566	45 830	56
Clinton	10 424					19 894	43
Methuen	4 814	Lawrence	44 654	Andover	6 142	184 363	48
Chelmsford	2 695	Dracut	1 996	Tyngsborough	(*a*)	87 782	87
Chelsea	27 909	Danvers	7 454	Everett	11 068	675 360	43
Southborough	2 114	Framingham	9 239			29 828	25
Holliston	2 619	Ashland	2 532	S. Framingham	(*a*)	19 926	83
Wellesley	3 600	Framingham	9 239			24 017	45
Merrimac	2 633	Newbury	1 427			27 805	40
Watertown	7 073					50 159	36
Needham	3 035	Boston ($\frac{1}{25}$)	22 423			56 910	19
Walpole	2 604					13 460	52
Boston ($\frac{1}{25}$)	22 423					39 739	48
Williamsburg	2 057					21 442	84
Medford	11 079					29 439	27
						157 594	4
Milton	4 278	Boston ($\frac{1}{25}$)	22 423			54 290	49
Billerica	2 380					7 681	43
Weymouth	10 866	Whitman	4 441			24 780	61
						28 738	19
						9 739	56
Framingham	9 239	Ashland	2 532	Hopkinton	4 688	26 358	43
Easton	4 493	Raynham	1 340			58 575	16
Reading	4 088	Saugus	3 673	Melrose	8 519	29 417	38
Brookfield	3 352	W. Brookfield	(*a*)	N. Brookfield	3 871	21 651	44
Grafton	5 002	Northbridge	4 603			17 213	16
Northborough	1 952	Westborough	5 195	Marlborough	13 805	107 056	11
Leicester	3 120	Spencer	8 747			100 950	32
						1 827 517 (*d*)	49
						2 352 765	43

s 41. Also—

Bellingham	1 334
Medway	2 985
	4 319

s 59. Also—

Spencer	8 747

* Estimated.

(*a*) Where populations are omitted, the census made no return; for approximation 500 inhabitants are assumed in each case.

(*b*) This total is 24 615 less than the column foots up, due to repetition of several cities in the table.

(*c*) If *s* 12 were omitted, this average would be 51.

(*d*) This total is 631 546 less than the column foots up, due to repetition of several cities in the table.

Mr. Davis.

TABLE No. 29.—Population Served by Roads

Name of Railway.	Reference No.	Order according to population served, smallest being 1 (Fig. 17)	Place.	Population served.	Place.
Maine.					
Bangor St. Ry.	C 1	15	Bangor	19 103	Brewer
Bath St. Ry.	C 2	3	Bath	8 723	
Calais St. Ry.	C 3	6	Calais	7 200	St. Stephens (N.B).
Portland R. R.	C 4	25	Portland	36 425	Deering
Total and average	...	.....	.....	.....	.....
Connecticut.					
Bridgeport Traction	**C 5**	27	Bridgeport	48 866	Southport
Hartford St. Ry.	**C 6**	33	Hartford.	53 230	Withersfield
Middletown St. Ry.	C 7	8	Middletown	9 013	Portland
New Haven St. Ry.	**C 8**	31	New Haven.	81 298	East Haven
New London St. Ry.	C 9	9	New London	13 757	Ocean Beach
Stamford St. Ry.	C 10	10	Stamford	15 700	
Winchester Ave. (N. Haven)	**C 11**	34	New Haven	81 298	Savin Rock
Waterbury Traction	C 12	22	Waterbury	28 648	Waterville
Total and average	...	.....	.....	.....	.....
Massachusetts.					
Boston Elevated	**C 13**	35	Boston	448 477	Cambridge
Braintree	C 14	20	Braintree	4 848	Quincy
Brockton	**C 15**	26	Brockton	27 294	Whitman
Commonwealth Ave. (Newton)	C 16	17	Newton	24 379	
Framingham Union	C 17	5	Framingham	9 239	
Gardner Electric	C 18	2	Gardner	8 424	
Globe (Fall River)	C 19	30	Fall River	74 398	
Gloucester	C 20	18	Gloucester	24 651	
Holyoke	C 21	29	Holyoke	35 637	Chicopee
Pittsfield Electric	C 22	13	Pittsfield	17 281	Dalton
Plymouth & Kingston	C 23	4	Plymouth	7 314	Kingston
Springfield	C 24	28	Springfield	44 179	W. Springfield
Taunton	C 25	19	Taunton	25 418	
Union (New Bedford)	C 26	24	New Bedford	40 733	Fairhaven
Wellesley & Boston	C 27	16	Newton	24 379	
West Roxbury & Roslindale	C 28	21	Boston ($\frac{1}{20}$)	22 423	Dedham
Worcester Consolidated	C 29	32	Worcester	84 655	
Woronoco	C 30	7	Westfield	9 805	
Total and average	....	.....	.....	.....	.....
" " (C 13 omitted)	....	.....	.....	.....	.....
New Hampshire.					
Union St. Ry.	C 31	12	Dover	12 790	Somersworth
Nashua St. Ry.	C 32	14	Nashua	19 311	Hudson
Concord St. Ry.	C 33	11	Concord	17 004	
Manchester St. Ry.	C 34	23	Manchester	44 126	
Laconia St. Ry.	C 35	1	Laconia	6 143	
Total and average					
Grand total and average (**C** 13 omitted)					

C 5. Also East Bridgeport .... (*a*)

C 6. Also East Windsor 2 890
Hockanum ..... (*a*)
N. Glastonbury } ...
S. Glastonbury } 3 457
Burnside ..... (*a*)
Newington ..... (*a*)
New Britain 16 519
S. Windsor 1 736
E. Hartford 4 455
30 557

C 8. Also Centerville ..... (*a*)

C 11. Also Milford 3 811
Orange 4 537
8 348

C 13. Also Newton 24 379
Somerville 40 152
Arlington 5 629
Brookline 12 102
Watertown 7 073
Medford 11 079
Malden 23 031
123 446

IN TABLE NO. 4. ELECTRIC CITY RAILWAYS.

Mr. Davis.

Population served.	Place.	Population served.	Place.	Population served.	Population served.	Passenger trips per capita per annum of population served.
4 193	.....	.....	.....	.....	23 296	65
.....	.....	.....	.....	.....	8 723	51
2 000?	Hilltown (N. B.)	(*a*)	.....	.....	9 790	49
5 353	Westbrook.....	6 632	.....	.....	48 410	112
.....	.....	.....	.....	.....	90 219	87
(*a*)	Westport	3 715	Fairfield.....	3 868	57 449	71
2 271	Windsor	2 954	W. Hartford...	1 930	90 942	98
4 687	.....	.....	.....	.....	13 700	29
(*a*)	Morris Cove.....	(*a*)	Westville.....	(*a*)	83 298	37
(*a*)	.....	.....	.....	.....	14 257	46
.....	.....	.....	.....	.....	15 700	39
(*a*)	City Point.....	(*a*)	W. Haven.....	(*a*)	91 146	48
(*a*)	Naugatuck.....	6 218	.....	.....	35 366	71
.....	.....	.....	.....	.....	401 858	61
70 628	Chelsea.....	27 909	Everett.....	11 068	680 928	266
16 723	Randolph	3 946	Holbrook.....	2 474	27 991	34
4 441	Stoughton.....	4 852	Holbrook	2 474	48 884	139
.....	.....	.....	.....	.....	24 379	59
.....	.....	.....	.....	.....	9 239	68
.....	.....	.....	.....	.....	8 424	40
.....	.....	.....	.....	.....	74 398	89
.....	.....	.....	.....	.....	24 651	55
14 050	Northampton.....	14 990	S. Hadley.....	4 261	68 938	61
2 885	.....	.....	.....	.....	20 166	65
1 659	.....	.....	.....	.....	8 978	77
5 077	Chicopee	14 050	Longmeadow..	2 183	65 489	177
.....	.....	.....	.....	.....	25 448	44
2 919	Dartmouth.....	3 122	.....	.....	46 774	80
.....	.....	.....	.....	.....	24 379	29
7 123	.....	.....	.....	.....	29 546	49
.....	.....	.....	.....	.....	84 655	125
.....	.....	.....	.....	.....	9 805	48
.....	.....	.....	.....	.....	1 215 794 (*b*)	193
.....	.....	.....	.....	.....	581 668 (*c*)	93 (*d*)
6 207	.....	.....	.....	.....	18 997	31
1 092	.....	.....	.....	.....	20 403	46
.....	.....	.....	.....	.....	17 004	57
.....	.....	.....	.....	.....	44 126	74
.....	.....	.....	.....	.....	6 143	37
.....	.....	.....	.....	.....	106 673	56
.....	.....	.....	.....	.....	1 180 418	78

***C* 15.** Also Easton..... 4 493
Randolph 3 946
Avon..... 1 384
9 823

(*a*) Where populations are omitted, the census made no return; for approximation, 500 inhabitants are assumed in each case.
(*b*) This total is 67 272 less than the column foots up, due to repetition of several cities in the table.
(*c*) Population served, omitting ***C* 13.**
(*d*) Rides per capita per annum of population served, omitting ***C* 13.**

Mr. Davis. the basis of estimates, and how much care must be taken not to be led astray in any given case. If we assume that the steam roads of the New England States serve the entire population, we get gross receipts per head, equal to $7.32, and they undoubtedly do serve this population, but if we were about to build the Boston and Albany Railroad (assuming it did not exist), could we properly use this figure, multiply by the total population and say that would be the probable receipts; or should we take the population within the limits of division (1) "those who live adjacent to the line and are compelled to ride upon it" and multiply by some higher figure, determined from other similar cases? The latter appeals to one as likely to produce much more accurate results. Table No. 16 gives the various amounts per head per annum for various assumed "populations served," from urban communities having, according to the United States census, populations of 8 000 and upward, to and including towns of 1 000 and upward. These divisions of population are not advanced as the proper ones, for they are not, but to bring out clearly what has been said before.

Turning to Tables Nos. 28 and 29, we see great fluctuations in the rides per capita per annum; this is partly due to many causes, such as population, density, distribution, etc., etc., but also due to inaccuracies in determining "population served." From the tables it is seen that the entire populations of towns are repeated for several roads and are considered as being served by and serving these roads; this is necessary unless we can divide the territory, which would be most difficult. If, however, it were done, and it can be, the results would be most interesting and instructive. By the divisions we have made, it is shown clearly that suburban and interurban roads either do not serve as great a population as the aggregate of those forming their terminals and through which they pass, or else the receipts per capita per annum are less from those they do serve. As a matter of fact both propositions are probably true. Our average of 78 in Table No. 29 would have been increased materially had road *C* 13 (Boston Elevated) been included with its 266 rides per capita per annum, but the result would not have been as accurate for application to other problems.

Effect of Competition.—One often hears of the competition which electric parallels have brought to our steam-railroad systems. This has been exaggerated greatly, for most of the traffic of electric railways did not exist until created by low "total cost" and frequent and quick service, although, in certain isolated cases, the building of electric parallels has temporarily drawn away traffic from steam railroads, only to be recovered as the total volume naturally increased. This fluctuation and recovery in traffic, on parallels which changed motive powers, has been shown clearly in the building of elevated and street railways in New York City. The Third Avenue Elevated so decreased the traffic on

the horse surface road as to cause alarm to the stockholders; the conversion of the horse railway to a cable road decreased the travel on the elevated, which was subsequently more than recovered. Mr. Davis.

Fig. 14 gives a good example of the decrease in traffic on the Manhattan Elevated Railway in New York City, operated by steam locomotives, due to the increase in speed and frequency of service on the Metropolitan Street Railway in its changes from horse traction to cable and electric. The loss on the elevated road from 1893 to 1897 was approximately 40 000 000 passengers—part of which was due to the financial depression throughout the United States, as indicated by the "dip" in all curves on Fig. 15; how much this amounts to it is impossible to determine and most difficult to estimate, but an approximation can be made from the retardation of increase shown in the curve on Fig. 15, giving passenger trips on all electric roads in Massachusetts. Projecting the curve by connecting 1893 with 1897 it would indicate a natural proportional increase in 1894 of 17 000 000 more passengers than actually took place, which represents approximately

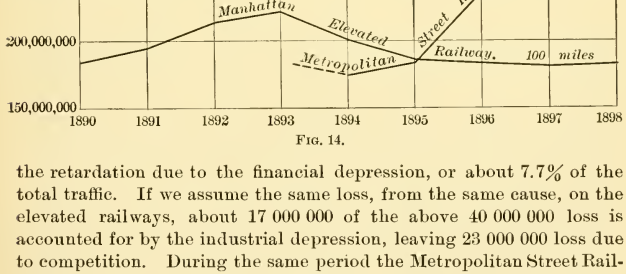


Fig. 14.

the retardation due to the financial depression, or about 7.7% of the total traffic. If we assume the same loss, from the same cause, on the elevated railways, about 17 000 000 of the above 40 000 000 loss is accounted for by the industrial depression, leaving 23 000 000 loss due to competition. During the same period the Metropolitan Street Railway gained about 110 000 000 passengers, or nearly five times as many as the elevated roads lost. The mileage of both roads remained constant, or nearly so, as in the Metropolitan curve are included, for the years taken, all roads now owned or operated by this company. These years also cover the change in motive power of this system. This increase in traffic on the street railways of New York undoubtedly comes largely from increase in speed, better physical conditions, such as track, cars, lighting, heating, cleanliness, open cars, etc., etc.

The elevated railways are operated under the same general conditions as exist on street railways, although the units are heavier, but they are very frequent; of course, they have the disadvantage of being

Mr. Davis.

TABLE No. 30.—EFFECT OF COMPETING ELECTRIC INTERURBAN PARALLELS TO STEAM RAILROADS.

Localities Connected.	Loss due to trolley parallels, as claimed by V. P. Hall of N. Y., N. H. & H. R. R. before Committee, State Legislature of Connecticut (*a*).	Approximate distance in miles from Railroad Commission Map.	Trips per day as given by time table of N. Y., N. H. & H. R. R.	Trips per day of trolley roads.	Number of passengers carried by Electric Railway System in and between these towns, part of which traveled between them (1894).
Norwalk—Rowayton	50%	4.75	27	84	956 241
Bridgeport—Stratford	$35 per day.	3.00	36	84	4 659 322 (Est.)
Bridgeport—Southport	80%	5.50	23	63	
Waterbury—Naugatuck	90%	6.00	13	69	2 624 421
Wallingford—Meriden	30%	5.50	17	30	2 001 347
Birmingham—Ansonia	90%	3.00	16	112	1 033 977
Winnepauk—S. Norwalk	(*b*) 50%	3.00	....	....	1 090 263
					12 365 571

(*a*) Total loss to N. Y., N. H. & H. R. R. from all parallel trolley roads in the State of Connecticut = $4 000 per month = $48 000 per annum, or 1/2 of 1% loss on total passenger income of $12 971 000 in 1894, as shown by Railroad Commission Reports.

(*b*) 64 passengers were carried on N. Y., N. H. & H. R. R. in the month of December, 1893, and 9 in the same month of 1894, or a total loss of 780 passengers per annum, at a possible maximum of 15 cents = $117.

confined to what might be called "trunk" lines without feeders. These frequent units are now operated by steam locomotives, but a change to electric motors is about to take place. This change is not warranted by any decrease in operating expenses which will take place, either actual or sufficient to offset the interest on the additional investment, although in the first years of electric operation figures will no doubt be produced which will appear to indicate such a result, as in two cases already cited. Nevertheless, the change, if made, will be a profitable one from the natural and induced increase in the traffic of the future, just as the large investments of our steam railroads in improved terminals, track, rolling stock and stations have been justified. Similar results will be the inducements for a change of operative methods on parts of our steam railroad systems, which change necessitates a change in motive power.

Table No. 30 has been prepared from a speech by Mr. Edwin B. Gager before the Railroad Committee of the State Legislature at Hartford, Conn., March 22d, 1895.* For many years bitter warfare has been waged against interurban electric railways by the Consolidated System (New York, New Haven and Hartford Railroad Company) resulting in the electric parallel law, where "public convenience and necessity" must be demonstrated to the satisfaction of the Superior

* Those who are interested can obtain a reprint of Mr. Gager's speech by addressing him at Derby, Conn.

Mr. Davis.

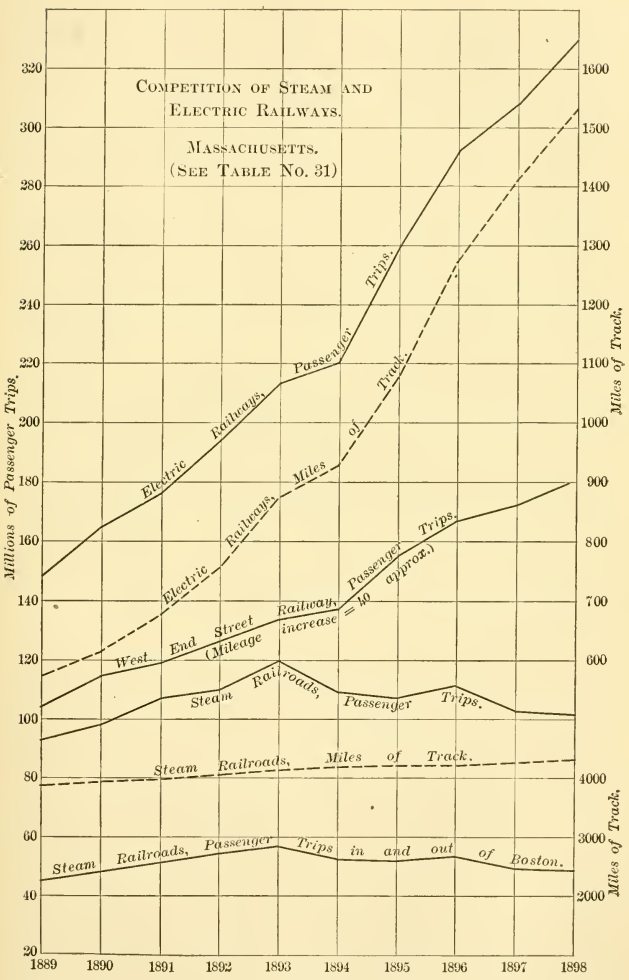


Fig. 15.

Mr. Davis. Court, before an electric railway can be built between two points already connected by a steam railroad. More unwise legislation against a natural progress, which would also benefit those whose influence created it, can scarcely be imagined. It is fair to assume that in this controversy—for the street railways naturally opposed such legislation—both sides produced the strongest arguments in support of their respective contentions; the Consolidated presenting losses of traffic, while the street railways insisted that their passenger travel was mostly an induced one, which did not and could not exist under steam-railroad conditions and operative methods.

An examination of Table No. 30 shows conclusively how the steam railroads convicted themselves. The Consolidated System only claimed a total loss of $4 000 per month, or $48 000 per annum on the entire system, being about $\frac{1}{3}$ of 1% of their gross passenger revenue. If the average fare were 10 cents, this would mean a total loss of 480 000 passengers per annum out of a total of 44 448 324, or 1.1%; but 1894 was the year of financial depression, when the steam railroads of Massachusetts lost 8.3% of their former passenger traffic, so that only part of this loss on the Consolidated was due to trolley parallels. While the total loss to the Consolidated was given by its officers, all the towns between which it occurred were not stated, so that in Table No. 30 the 12 365 571 passenger trips, between and in a few of these towns, is only part of the total passenger traffic of the street railways serving all localities where such loss took place. Whether this figure should be increased by 50 to 100%, or more, we cannot say, but, in any case, the data are sufficient to show the large induced traffic of street railways; or, in other words, systems which give low fares, frequent service, short total time consumed in round trip and a "leave-at-your-door" service.

To emphasize the fact of what might be called a "dormant traffic," which can become an "induced traffic" under proper operative conditions, and to bring out this fact more clearly, Fig. 15 and Table No. 31 have been prepared, and these again show what a small part of the traffic of street railways has come from the losses of steam railroads, and also, that a large part of this loss has been wrongly attributed to electric parallel competition.

The passenger traffic on steam railroads in Massachusetts has increased constantly from year to year since 1870, except during two periods, both of which coincide with industrial depressions; this is shown by Curve 2, Fig. 7, where the loss began in 1873 and 1893. Short-haul passenger traffic is but little affected by financial conditions, when compared with the effect on long-haul traffic; this is due mainly to necessity being the basis of short-haul passenger business, or, in other words, it is composed mostly of commuters or suburban and interurban travel. Curve 2, Fig. 6, corroborates this position,

Mr. Davis.

for we see that the passenger trips on street railways (short hauls, from necessity, as the controlling factors in volume of traffic on these systems), increased after 1874 and 1893, although less rapidly than in the year previous, but there was no actual decrease. This curve brings out another interesting fact, namely, the quicker recovery of electric roads and their more rapid increase in passenger traffic than when operated by horse-power, again supporting our position. In Massachusetts, electric railway mileage has increased along with the passenger traffic (1889 to 1898) 163% in the former to 123% in the latter, or, approximately, each has kept pace with the other. In other words, these railways have been built where traffic did not previously exist, nor could it be produced by the steam railroads under existing conditions; it has been "induced" by the character of the electric roads and their operative methods. Steam railroad mileage has only increased 10% in Massachusetts during the same period, and passenger traffic 9.6% (net).

TABLE No. 31.—Comparative Loss on Steam Railroads and Gain on Electric Railways—Massachusetts.—(See Fig. 15).

Year.	Loss in passenger trips per annum on all steam railroads in Massachusetts.	Loss in passenger trips per annum on steam railroads in and out of Boston.	Loss in passenger trips per annum on steam railroads in Massachusetts omitting those in and out of Boston. Column 2 minus Column 3.	Gain in passenger trips per annum on all street railways in Massachusetts.	Amounts in Column 5 divided by those in Column 2.	Gain in passenger trips per annum on West End Street Railway system in and out of Boston.	Amounts in Column 7 divided by those in Column 3.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1889 to 1893....	{ 26 250 648 } { Increase }	{ 11 406 434 } { Increase }	{ 14 844 214 } { Increase }	{ 65 362 606 } { Increase }	2.4	29 620 468	2.6
1894.....	10 345 763	3 824 940	6 520 823	6 912 090	0.65	3 164 831	0.8
1895.....	1 577 836	740 680	837 156	39 330 209	24.9	18 203 057	24.5
1896 (Increase)	.....	.....	.....	.....	.....	.....	.....
1897.....	8 885 161	3 808 480	5 076 681	16 325 281	1.8	5 692 225	1.4
1898.....	803 168	723 748	79 420	22 205 405	27.6	8 766 782	12.1
1896 (Increase)	21 611 928	9 097 848	12 514 080	.....	.....	.....	.....
	3 772 703	1 483 774	2 288 929	32 564 635	8.6	11 630 782	7.8
Net decrease 1894 to 1898.	17 839 225	7 614 074	10 225 151	117 327 620	6.5	47 457 677	6.2

Turning to Table No. 31 and Fig. 15, we see that the net loss of passenger traffic on all the steam railroads of Massachusetts, from 1893 to 1898, was 17 839 225; but during the same years the street railways increased 117 327 620, or 6.5 times the loss of the steam railroads. If 7.7% of the traffic was lost, due to financial conditions (as estimated heretofore), then only 8 600 000 passenger trips were lost to

Mr. Davis. the steam roads of the State from trolley competition. If the average fare lost was 10 cents (when secured by the street railway the fare would be halved or even less for them), the total amounted to $860 000; or 2.4% on the gross passenger earnings and 1.1% on the gross earnings of the steam railroads; this would only amount to about 1% of the total net earnings. Again, the net loss to steam railroads in and out of Boston, from 1893 to 1898, was 7 614 074, while the West End Street Railway (controlling practically all street railways in and out of Boston), increased 47 457 677 or 6.2 times the loss of the steam roads. Applying the same argument, only 3 225 000 passengers were lost by the steam roads in and out of Boston, due to this competition. This would not be fair, however, for this traffic is made up more largely of commuters than long-haul passengers. Assume it at 5 000 000, which is undoubtedly too high, and with an average commutation rate of 7 cents, the loss is only $350 000 at the maximum. Furthermore, the greatest gains of the street railways throughout the State, and the West End Street Railway alone, were in 1895 and 1898, when the steam roads lost the least. All these data point to the conclusion, already stated, that competition does not take place to the extent usually believed; while an "induced" traffic is created by the low fares, frequent, quick and "leave-at-your-door" service, rendered possible by the physical characteristics and operative methods of the electric roads.

Fig. 16 is from Table No. 28. Populations are arranged in the order of magnitude as abscissas, beginning with the smallest served, while the passengers per capita per annum are ordinates. The resulting curve shows no well-defined law.

Fig. 17 is the same from Table No. 29, and indicates plainly that as populations increase, the rides per capita per annum also increase.

Fig. 18 is made from towns arranged in the order of their density of population. The curve shows no well-defined law.

The determination of the desirability or undesirability of substituting electric traction for steam on parts of some American railroad systems is a matter of constantly increasing importance; on many projected lines the question whether steam or electricity shall be used is a problem which must now be solved, although in the past there was no question as to which should be adopted. We will probably see during the next ten years a large increase in the substitution of electricity for steam on railroads. This change is foreshadowed by the present electrical equipment of elevated railways in Chicago; by the overhead-trolley and third-rail experiments of the New York, New Haven & Hartford Railroad Company; by the operation of several branch lines of the Pennsylvania Railroad Company with electric motors; and by the present active discussions, among steam-railroad engineers and managers, of the details of first cost, operating expenses, and methods of electric traction.

Mr. Davis.

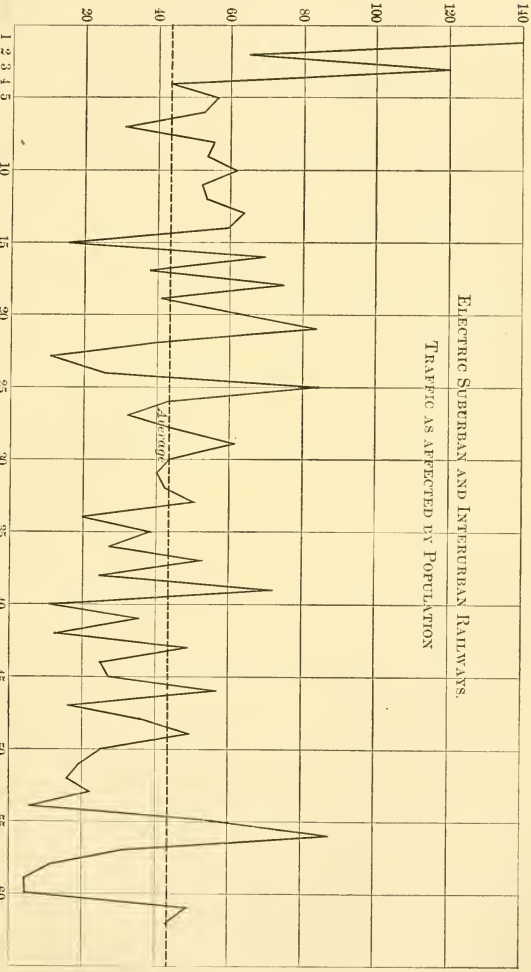


Fig. 16.

Mr. Davis. If a change is contemplated, steam-railroad managers must avoid making the mistake which took place in the change from horse traction to electric traction—namely, of trying to reduce the first cost of changing by the use of old methods, material, and equipment, which, although entirely suited to the old system, proved most unsatisfactory under the new conditions. The old equipment partly made over will not do. There must be new trucks, and new and lighter car bodies, hung lower for greater ease of entrance and departure. Old methods of operation must be discarded and new ones substituted. A change of system may necessitate additional tracks, which should be provided even at large cost. The nearer the approach to the "leave-at-your-door" service, the greater the success.

Let us consider some examples where it is probable that electricity could be substituted with profit, remembering that more thorough investigation might show, in this or that case, that the change is undesirable.

Suburban service of the Pennsylvania Railroad, out of Philadelphia, as follows:

P., W. & B.	Division to	Wilmington.
Main Line	"	" Paoli.
Germantown & Chestnut Hill	"	" Chestnut Hill.
Reading	"	" Norristown.
U. R. Rs. of N. J.	"	" Tacony.
West Chester	"	" West Chester.

By the use of multiphase currents and rotary-transformer sub-stations, power houses could be located along the Delaware and Schuylkill rivers; from these sub-stations direct current at 500 volts could be fed into the trolley or third-rail line. The equipment should be entirely new; cars of the same seating capacity, but lower and of lighter construction, should be used: the bridge over the Schuylkill River into Broad Street Station should be widened by four additional tracks; main line and U. R. Rs. of N. J., by two tracks, each; P. W. & B. should have a total of four tracks; Germantown & Chestnut Hill, Reading, and West Chester should each have a total of three tracks: all tracks should be equipped electrically, although some of them would be used by steam locomotives hauling freight and through passenger trains, and some or all, at times, by both systems. Two tracks should be laid in the street from Broad Street Station to the Schuylkill Bridge coming to railroad grade at that point: the cars running over these tracks should operate from the Delaware River out Market Street, by trackage arrangement with the street railway or by special franchise, the street being wide enough for four tracks: they should be run as any street-railway line is run to-day, and continue out on the extra tracks, provided above, as far

Electric City Railways.
Traffic as affected by Population.
Passenger Trips per Capita per Annum.
260
240
220
200
180
160
140
120
100
80
60
40
20
Average
5
10
15
20
25
30
35
Order of Magnitude
According to Population served by Railways in Table No. 29.

Fig. 17.

Mr. Davis. as circumstances might warrant, and should be extended as travel required; right-of-way should be prepared, so that passengers could leave or enter these cars at any point along the line. Local stations should be provided, in some cases much closer than at present, at which passengers, on the cars mentioned above, could change to electric cars stopping only at said stations, or to express electric trains stopping at fewer points, all being done by the payment of a single fare. It will be seen that this is a combination of the street railway on the outside tracks with the local and express service of a steam road on the inside tracks, while the through locomotive trains operate on the middle tracks. The proposition is a radical one, is advanced as such, and will bear close investigation and study; the expense would be enormous, but the gross receipts from such a system of street railways, accumulating passengers for the rapid-transit system connected so intimately with it, would also be enormous—in fact, the writer believes, far in excess of anything yet accomplished in the transportation of passengers.

The same treatment of the following terminals, with modifications to suit each case, would be worth studying:

New York, New Haven and Hartford Railroad, out of Boston.
Boston and Maine, out of Boston.
Philadelphia and Reading, out of Philadelphia.
Harlem, New York Central and Hudson River, and New York, New Haven and Hartford Railroad Companies, out of New York City.
Central Railroad Company of New Jersey, out of Jersey City.
Illinois Central Railroad, out of Chicago, etc., etc.

The above is, of course, only a suggestion of details.

As an example of interurban traffic, we can take Ansonia and New Haven, Conn. When the steam railroad owns the systems of street railways in both towns, their cars will pick up passengers at either center, will pass onto the present steam tracks on the right-of-way of the New Haven and Derby Railroad (New York, New Haven and Hartford Railroad Company), run at high speed without stops to the other center, pass onto the local street railway tracks there and distribute its passengers where they desire, all for one fare. Such a system operated by electric motors would be a financial success, where a line like the third-rail between Hartford and New Britain is a failure in the true sense. Many other similar examples might be given, but this indicates the future of our steam railroads.

It must not be inferred that all or any of the cases suggested above, if thoroughly investigated, would prove the expense warranted, especially that of laying additional tracks. It is, however, the writer's judgment that in most cases additional tracks would be necessary to

make the system profitable and able to handle the "induced" traffic. Emphasis must be placed upon the importance of radical changes in present steam-railroad methods to make the change to electricity pay. Electric traction in connection with existing tracks and equipment, and under existing operative methods, will result in heavy loss and ultimate abandonment of the change—possibly even in disaster. Mr. Davis.

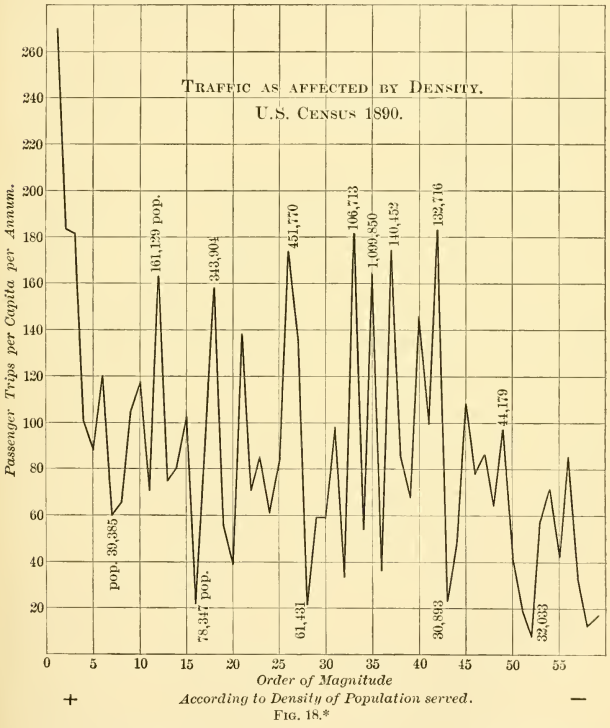


Fig. 18.*

The argument has thus far been confined, for reasons stated, to suburban and interurban short hauls, and the writer feels convinced from his investigation of the subject that the adoption of electricity as a mode of operation, with the additional change to the prevailing

* The data for the diagram, Fig. 18, are taken from Table No. 39 of "Facts and Figures Interesting to Electric Railway Men" (3d edition), published by the writer.

Mr. Davis. methods of our present street railways giving nearly the same class of service (combined or not with steam locomotives as the case may require), will result to the steam roads in an enormous increase of the total passengers carried, passengers per car-mile and rate received per mile, and will shorten the length each passenger is hauled; and, by thus increasing the gross receipts, will more than pay for the additional investment. If this cannot be done in any given case, then it will not pay to make the change.

In conclusion, the writer wishes to quote from the final paragraphs of a paper presented by him to the American Institute of Electrical Engineers,* wherein was expressed the convictions given in this discussion, and which he sees no reason to change at this writing. This was prior to the third-rail experiments of the New York, New Haven and Hartford Railroad.

"The question whether the gross receipts of a given road will be affected by the use of one or the other power under discussion is a most interesting one. Experience shows that where an electric road has paralleled a steam road it has taken most of the latter's business at first, but less as time went on; and that it created a demand for intercommunication which had never existed before—the bulk of the passenger travel coming from this cause. This is interestingly shown in the arguments of Judge Hall (Vice-President of the New York, New Haven and Hartford Railroad) and Judge Gager, of Connecticut, before the Legislature of the State at its last session. This, of course, refers to passenger receipts only. Freight receipts would not be affected by the use of one power or the other, they increasing only as the country grows and rates fall, together with better facilities. Receipts from express and mails might be materially increased by the use of electric traction when giving more frequent service. It appears that the close headway and 'leave-at-your-door' service of electric roads are the main reasons for their induced travel. The question of how much more the travel would be increased by the use of electric traction and frequent service is problematical, for the 'leave-at-your-door' service is wanting in steam railroads as they are, but why not change them? If this could be done, past experience and data would give a good basis from which to estimate future results.

"The conclusion one arrives at is that for long lines, infrequent service, where freight is a large proportion of the business, and where centers of population are far apart, the steam locomotive is the only paying method of to day, as the first cost will be less, as well as total expenses. The writer has had several opportunities of determining these facts. What development may bring to electric traction in the far future cannot be foretold.

"The writer believes that electric traction will be profitable to steam railroad systems when some or all of the following conditions are fulfilled, depending upon the special problem to be solved:

"1. Steam railroad managers must avoid making the mistake which took place in the change from horse traction to electric traction, namely, of trying to reduce the first cost of changing by use of old methods, material and equipments, which, although entirely suited to

* Meeting of October 21st, 1896.

the old system, proved most unsuitable under the new conditions. The tendency is to repeat this mistake, and too much stress cannot be laid upon avoiding it. The old equipment partly made over, the old method of operating, etc., will not bring success in the use of electric traction; and if followed from necessity would indicate the strongest argument against the change. Mr. Davis.

"2. Long distance, heavy trains and infrequent service, if a necessity, will prevent electric traction being profitable. Therefore, where gross receipts can be increased by light trains and frequent service, and thus decrease expenses, as compared to steam locomotives, electric traction will prove profitable. One of the best examples of how this could be applied is found in the suburban service of the Pennsylvania Railroad out of Philadelphia. It is, of course, understood that electric cars can be operated over the same tracks as trains drawn by steam locomotives; a change of system requiring more frequent service for success might necessitate one or more additional tracks, which in some cases would delay the time when a change would be advisable.

"3. Steam railroads, where the second condition is fulfilled, can better the results where they operate part of the system on the 'leave-at-your-door' plan. This suggestion may seem to some a radical departure, but I commend it to the careful thought of those interested."

The writer's final conclusion is that electric traction can be profitably used:

(1) Where units are so light and so frequent, and carried such short distances, that steam locomotives cannot be furnished at lower first cost or operated more cheaply, as in the plain case of a street railway.

(2) Where the gross receipts will be so increased by the change of system and the mode of operation as to more than pay for the increased first cost and operating expenses per car-mile, as in the case of suburban steam systems running out of our great cities and interurban systems between large towns situated a few miles apart.

(3) When competition of parallel electric roads compels the change to save what traffic there is, irrespective of the question whether it will be more profitable.

(4) Where higher speeds are required than can be attained by steam locomotives.

High Speed Railroads.

This discussion would not be complete without reference to another field where electric motors will prove a profitable method of operation, namely, (4) where higher speeds are required than can be attained by steam locomotives. This discussion is already long enough, so that the consideration of this must be postponed, but the reader is referred to "Enormous Possibilities of Rapid Electric Travel," by the writer.*

* *Engineering Magazine*, 1897.

Mr. Davis. In that paper is expressed the belief that large centers of population, reasonably near together, will eventually be connected by railways operating at 150 miles per hour or over, with the local street railways as feeders and distributors.

Many more interesting facts and deductions can be obtained from a careful examination and analysis of the curves and tables. They are left to the intelligence of the reader.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

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No. 863.

THE REACTION BREAKWATER AS APPLIED TO THE IMPROVEMENT OF OCEAN BARS.

By LEWIS M. HAUPT, M. Am. Soc. C. E.

PRESENTED OCTOBER 18TH, 1899.

WITH DISCUSSION.

RESULTS TO DATE.

In a paper published by the Society in the *Transactions** for December, 1898, attention is called to the single curved breakwater at Aransas Pass, Texas, designed and located by the writer for the purpose of creating at that inlet a navigable channel at least 20 ft. in depth.

In the discussion which followed that paper, it appeared that the incompleted breakwater was doing effective work; but as no survey had been made for some time, and the Harbor Company had suspended all work since May, 1897, it was impossible to state the results with any degree of accuracy. The latest examination available at that date was that made under the direction of a Board of Engineers for the purpose

NOTE.—Additional discussions on this paper, received since it was prepared for *Transactions*, will appear in the *Proceedings* for January, 1900, and, together with any further contributions to the subject, will be published in Vol. XLIII of *Transactions*.

* "Origin of the Gulf Stream and Circulation of the Waters in the Gulf of Mexico, with Special Reference to the Effect on Jetty Construction," by N. B. Sweitzer, Jr., Jun. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. xl, p. 86.

of appraising the value of the work done, with a view of turning it over to the Government to continue. This board reported on November 22d, 1897, that:

"The depth has not been materially, if at all, increased." * * * "At the time of examination of the Board, in July, 1897, it was 8.8 ft., and in November, 1897, 8.5 ft. The depths and contour lines are shown on appended charts."

(These charts, however, show a least depth at one crossing on the crest of the bar of 9¼ ft., November 2d to 5th.) The Board also expressed the opinion that "there does not seem any probability that the jetty as now constructed will of itself secure and maintain any considerable increase of depth in a navigable channel of proper width."

In view of this expectation and the fact that up to the end of February, 1899, a period of 15 months, the unaided currents have increased the depth at this particular part of the bar from "8½ ft.," as reported, to over 22 ft., and that, notwithstanding the existence of a large part of the old Government jetty across the channel, the question may pertinently be asked, whence these phenomenal results in so short a time?

Reaction vs. Concentration.

The answer is plain, since the curved form of the breakwater (not "jetty"), is such as to develop the potential energy of the ebb currents, rendering them kinetic, and applying them locally on the crest of the bar where they are needed for scour. At the same time, the breakwater serves to arrest the littoral drift and defend the channel from its encroachments. In short, it aims to utilize the well-known centrifugal force by the reaction developed by the resistance caused by a continuous change of direction so often manifested in the concave bends of streams, instead of the principle of concentration which has been relied upon so largely in the system of twin or convergent jetties, with such meager results.

This may be well illustrated, perhaps, by the case of a projectile flying through the air with a large amount of stored energy, which is developed suddenly only when meeting with a point of resistance. It has also been observed that currents having a velocity sufficient to scour the material over which they are flowing do not disturb it, but if a resisting log, rock or other obstruction is introduced, the obstruc-

tion is undermined and often buried and covered up by the reaction which it develops in the currents.

In Ex. Doc. 78, 48th Congress, it is stated that "The mean ordinary velocity at the Narrows (New York Bay), is, during the ebb tide, about 2 ft. per second, and from this a depth of 100 ft. results." This does not follow, as there are numerous sandy bars having a much greater mean velocity over them which lie at depths less than 10 ft. from the surface. Manifestly, it is not velocity, therefore, that constitutes the working force to produce scour, but reaction, just as when impounded waters are dammed back prior to escaping through a breach, when the increased head and contraction react upon the bottom, affecting the entire volume and producing deep eddies. In the case of a single obstructing pier head, the effects will often extend to depths of 50 ft. or more, depending on the intensity and direction of the currents, scouring out the material and depositing it in the lee of the pier. In the case of the Narrows, above cited, it is not the ebb which scours, but the flood current, the resultant of which is near the bottom and which runs for eleven hours out of twelve up stream or into the Upper Bay, indicating a huge vertical eddy.*

The principle of concentration or trailing by convergent or parallel jetties, on the other hand, has been applied frequently by maritime engineers, but generally without producing the desired results unless aided by dredging. The most successful instances of this method are those at the mouths of large rivers which debouch into nearly tideless seas, as at the Sulina mouth of the Danube, built by Sir Charles Hartley; the South Pass, by the late James B. Eads, M. Am. Soc. C. E., and the Tampico jetties, by E. L. Corthell, M. Am. Soc. C. E.; but for purely tidal entrances the effect of the two jetties is to invert the natural trumpet-shaped opening and to diminish the area of the gorge, which is transferred to the crest of the bar, thus reducing the tidal volumes, preventing the complete filling of the interior compartment and requiring the same volume to move in both directions over the same path. The effect, therefore, is to minimize the movement and reduce the resultant to zero. Probably the most successful tidal harbor improved by convergent jetties is that of Dublin, where the improvement is largely due to a gap of 600 ft. left in the "Great North Wall," or jetty, near its shore end, to admit the flood tide

* Ms. Report of Professor Henry Mitchell, U. S. C. & G. Survey Office, or "Harbor Studies" by the author. Library, Am. Soc. C. E.

which passes over the North Bull Sands, and which flows out chiefly through the jetties because of the form of the basin and the great range of the tides, which is there about 13 ft.

The following extracts from a paper read May 20th, 1879, by John Purser Griffith, Assoc. M. Inst. C. E., on "The Improvement of the Bar of Dublin Harbor by Artificial Scour," will serve to illustrate this point :

"Mr. Giles recommended that an opening should be left at the shore end of the proposed wall 600 ft. wide, to allow of a free passage for the tidal waters north of the Green or Bull Island, as the sand island on the North Bull was called. This opening had been proposed by Captain Corneille in 1802; but at that time the Directors General of Inland Navigation feared that the flood tide flowing through this opening would carry sand from the North Bull into the harbor, and that the injury thus done to the port might more than counterbalance any advantage to be gained by the opening.

"In 1835 Sir William Cubitt reported upon the state of the harbor. Referring to the improvement of the bar, he says: 'The great increase of depth and improved channel over the bar I attribute entirely to the erection of the Great North Wall, a measure founded upon sound principles, and carried into effect in a manner well calculated to effect the desired purposes—viz., that of checking the influx of sand upon the flood tide from the North Bull into the harbor, and giving an increased impetus at the ebb tide by means of narrowing the stream, and confining it to a direction suitable for keeping open the best channel, the effect of which is already shown by an increased depth of 5 ft. over the east bar since the erection of the Great North Wall.'

"Captain Washington, in his report, written in 1845, on the harbor of Dublin, as one of the Tidal Harbors Commissioners, referred to the Great North Wall as follows: 'The propriety of this measure, which involved so heavy an expenditure, has been a subject much controverted amongst persons connected with the port; but it is believed that there is now but one opinion as to its beneficial effects, and that the plan evinced both good judgment and skill.'

"There are few, if any, similar cases of a bar and entrance channel to any harbor being increased in depth like that of Dublin, viz., about 7 ft. in thirty years, and I think great credit is due to those who designed, as well as those who executed works which have achieved so important a result.

"The improvement of Dublin Harbor entrance ranks second to none. There is no other example, so far as the author is aware, of the construction of an artificial estuary for scouring purposes which has proved so successful.

"In the discussion, Mr. Abernethy, Vice-President, said he thought

attention should be directed to the construction of works which would tend to increase the tidal volume, and at the same time to prolong the action of the outgoing currents from the period of half ebb towards low water.

"Mr. Bergeron stated it was very difficult, almost impossible, to dredge sand bars in an open sea.

"Mr. Vernon-Harcourt said that to diminish the tidal capacity within a harbor was the worst thing that could possibly be done.

"Mr. Griffith said: 'Prior to the construction of the Great North Wall the ebb of a spring tide attained a velocity of $1\frac{1}{4}$ miles per hour across the bar, while at present it reached nearly $3\frac{1}{4}$ miles, showing an increase of about 2 miles per hour. He did not believe that very high velocities were efficient. Several instances might be named in which high velocities were attained, and yet the scour was a failure.'

"Mr. Stoney thought that though there might be some doubt as to the authorship of the Great North Wall, there could be none as to its complete success."

From this quotation it appears that some of the best authorities in the world recognize an increase in depth of 7 ft. in thirty years as being beyond precedent, yet at Aransas Pass, with an incompleted reaction breakwater, having large gaps at both extremities, a feeble tide (only 14 ins.), and a serious submerged obstruction almost completely blocking the channel, the depths have progressively increased from about 6 ft. to an average of over 18 ft. in a few years, while in some places there are depths of 23 ft.; so that instead of a deepening at the rate of 1 ft. in over four years, the figures are reversed and show over 4 ft. in one year, a rate which is about twenty times greater than the best heretofore recorded.

These results are the more remarkable as they have been obtained without the aid of dredging, and on a bar which is composed of hard, fine sand, so compact that a boat hook or the blade of an oar does not penetrate it; so solid is it, that a Board of Engineers, in 1889, said of this bar that "The necessary results could not be obtained without extensive and almost constant dredging, which would involve incalculable expense."

As early as 1871, the project of obtaining a navigable channel over the Aransas Bar was pronounced impracticable because of its cost. The report of the United States Engineers of 1871 stated that "The cost of building a jetty, from Mustang or St. Joseph's Island towards the bar, which would be able to resist the action of the storms upon the quicksand foundation, must be an insuperable objection to any

such experiment;" yet it was finally considered expedient, and it was further estimated in 1879 that, to obtain a 12-ft. channel would cost $759 185. (The ruling depths are 8 to 8½ ft.) Work was commenced on this project in 1880 and, after an expenditure of $550 416, up to June, 1890, it was discontinued, with a resulting depth of 7½ ft. In 1887 the project was modified "to secure and maintain a channel depth of not less than 20 ft.," and the same report stated that "The original estimated cost of this work, as here revised, is $2 052 543.72."

Cost of the Reaction Breakwater.

As the Government abandoned the work at Aransas and concentrated its appropriations on Galveston, a private company, known as the Aransas Pass Harbor Company, was incorporated, March 22d, 1890, and soon thereafter secured a large land bonus and proceeded to construct another jetty which, like its predecessor, was erroneously located on the southerly side of the Pass. It proved to be a failure, as was predicted, but seemed to be a necessary part of the experience of the Company.

At a later date the Company was reorganized, and it was then decided to give the reaction breakwater a trial by building only half of it, for which half depths of 15 ft. were predicted. Work was commenced in August, 1895, and in a few months the bar deepened to 13 ft.; but, unfortunately, the remains of the original Government jetty were then uncovered and found to be still in place, lying across the proposed channel, although previously officially reported to have "disappeared." The building of the breakwater to high water intersected this old work and formed a pocket which arrested the sand and prevented scour, while the Harbor Company failed to remove the obstructing jetty, as advised to do, until the winter of 1896–97, when a contract was made with Col. C. P. Goodyear to complete the work, he to advance all the funds and assume all risks.

He exploded some 23 350 lbs. of dynamite on the old jetty and bar, causing a breach of about 500 ft. in length near the breakwater, but being unable to secure the capital in time to continue the work, he was reluctantly obliged to surrender his contract in May, 1897. Since that time no work has been done. About ¼ mile of the breakwater near the center was built to full dimensions; the shore flank was extended to its minimum limits, but of a contracted section, with large

gaps through which the ebb currents escaped seaward and the sand entered the channel during flood tide, and the mat was laid at the outer end to a distance of 1 000 ft. All this work was done by Charles Clark & Company, of Galveston, and notwithstanding the storms, which have been violent, and the alleged quicksand, there is no appreciable loss or shrinkage in the sections of the structure, which has stood more than three years, in a very exposed position, doing excellent service.

The total length of the structure as designed is 6 200 ft., and about $250 000 has been expended on it.

The quantities were:

Rip-rap (core)	47 246	cu. yds.
Cap stones	9 183	"
Mattress work	17 000	"
	73 429	"

Say 75 000 cu. yds. at $3.33 per yard, and the cost per foot of depth gained, to date, has been approximately $25 000. It would, doubtless, have been much less but for the old jetty, which is still a serious obstruction and menace in the navigable channel.

As the total length of this work is less than half that required by the parallel-jetty system, it is evident that its cost will be much less while its efficiency is much greater, as it creates and maintains its own channel.

An official report of a thorough survey made in January and February, 1899, by the United States Coast and Geodetic Survey states, *inter alia:*

That during the winter the weather was so bad that it was only practicable to get on the bar for survey purposes about one day out of each week, and that "very heavy gales prevailed with terrific force with the wind from the north-north-east."

"In 1895 the Aransas Pass Harbor Company constructed a jetty in the shape of the letter S on the north side of the entrance, which is still in existence, and ever since its construction there has been a marked increase in the depth of water on the bar. The present channel crosses the Mansfield jetty, portions of which are still in existence. An attempt was made to remove this by explosion of dynamite, with the result that the rocks scattered over considerable area, and without doubt they prevent the current from scouring the bottom to its full capacity. * * * It is my opinion that by the completion of the

present jetty and the clearing away of the rock in the entrance that a channel of at least 20 ft. in depth would soon be secured."

This report, taken as a whole, confirms in every particular the predictions of the inventor as to the results to be expected and the features embodied in a breakwater of this peculiar form and position.

The conditions which this structure was designed to fulfill were:

1. That it should arrest the littoral drift of sand and defend the channel from its encroachments—hence it must be placed to "windward" of the proposed channel.

2. That it should freely admit the full tidal prism during the entire period of flood tide, that there might be no reduction of volume at ebb; consequently it was detached from shore, there being a gap of nearly 1 800 ft. left open at the inner end.

3. That it should catch and control the greater part of the discharge at ebb and confine it locally to a narrow path across the bar, thus changing the conditions of equilibrium of the tidal movements largely in favor of the ebb.

4. It should be a compound and reverse curve, concave to the channel, and have such radii as to produce the best dimensions of waterway consistent with the volume of discharge, so as to develop a continuous reaction entirely across the bar and at the same time afford a good navigable channel.

5. It should also be so adjusted, with reference to the axis of the discharge, as to receive the same tangentially and train it across the bar without creating abrupt points of incidence, causing inequality of depths, and, at the same time, not producing so much compression as to cause undermining and ultimate destruction or large expense for maintenance.

6. It should also control a sufficient volume to prevent any encroachment of drift from the opposite side of the channel during the season of change in the direction of angular wave movements, and thus construct its counterpart of sand to aid in controlling the currents, without cost.

7. It should not promote the growth of the bar seaward by the deposit of the material removed from the channel on the outer slope of the bar.

8. It should rise above high water to afford an aid to navigation by breaking the waves and indicating the position of the channel.

9. It should be so located as to develop the channel across that part of the entrance where the bar-building forces are weakest, thus giving quicker and more permanent results.

That this single, detached, reaction breakwater fully satisfies the foregoing requirements, even under adverse circumstances and in an incomplete state, is demonstrated sufficiently by the work done at Aransas Pass, as shown by the progressive deepening and enlargement of the channel since the breach was made in the old Government jetty by Colonel Goodyear in 1897. That it is a stable, permanent and economical structure is also shown by the condition of the work after tests by severe storms and by years of exposure. That it has a remarkable power of producing erosion is shown by the fact that in February, 1895, the distance across the bar between the 15-ft. contours on the inside and outside was 3 650 ft., or nearly $\frac{3}{4}$ of a mile, which has been cut through except a few hundred feet, giving a navigable channel at mean low water of over 14 ft., and without material advance of the bar seaward.

It is not doubted by competent authorities that if the breakwater had been completed in the ten months, as originally proposed, and the old jetty had been promptly removed when discovered *in situ*, the predicted 20 ft. would have been secured ere now, and without aid from dredging, while the channel would have been self-maintaining.

In short, this simple structure marks a distinct advance in the resources of maritime engineers in the improvement of ocean bars, with great economy in cost of construction and maintenance, and large saving of time. It is the result of about 25 years of research and over 11 years of effort to secure a practical demonstration, and it has now reached a point sufficient to prove its great utility.

It is but just to add that these unprecedented results were probably unknown to the Board of Engineers which stated in 1897 "that the value to the Government of the works of the Aransas Pass Harbor Company for the improvement of Aransas Pass, Tex., is nothing," and that the same Board in reporting on the same subject December 17th, 1898, therefore urged the removal of the outer 1 000 ft. of foundation; the construction of a sill 40 ft. wide and 3 ft. deep, which would have obstructed the tides by closing the opening at the shore end; the construction of a parallel straight jetty on the southerly side of the channel and other works, supplemented by the dredging of 750 000 cu.

yds. at an estimated cost of $1 525 000. This estimate was met by a proposition from a responsible firm of contractors to guarantee the 20-ft. channel within two years at a cost of less than half of the above sum, with the result that Congress concluded to appropriate $60 000 to remove the obstructing portion of the old Government jetty, and permitted the incomplete breakwater to remain unmolested, that it might continue to deepen the channel by natural agencies and so further demonstrate its utilities. At that date the reported depth was 13 ft. 4 ins., while since then it has increased, within a few months, to about 18 ft. at the same point, thus confirming the wisdom of this action. With the gaps closed and the structure completed, there should result, in a short time, the full depth of 20 ft., as predicted, and at a cost considerably less than half of that of the parallel jetties, as originally proposed. The manifest economies of this method should suffice to commend it for more general use in the improvement of bars on alluvial coasts where there is a resultant littoral drift.

The writer desires to acknowledge his indebtedness to H. C. Ripley, M. Am. Soc. C. E., and George Y. Wisner, M. Am. Soc. C. E., consulting engineers, for assistance in preparing the specifications and for their unshaken confidence in the results to be expected from this plan from its inception, as well as to Colonel C. P. Goodyear for his successful effort to breach the old jetty, and thus enable the partially controlled currents to reach the crest of the bar.

The work was done under the efficient supervision of William Dunbar Jenkins, M. Am. Soc. C. E., resident engineer for the Harbor Company, and his assistants, Messrs. Pitts and Collins.

APPENDIX.

THE THEORY OF THE REACTION BREAKWATER.

It was the purpose of the writer to submit only a brief statement of physical conditions and results secured under very adverse circumstances, but, in deference to the opinions of the Committee on Publications, a digest of the researches which led to the development of this form of breakwater is added to supplement the subject.

The numerous and conflicting conclusions, reported after careful instrumental surveys made to determine the direction and intensity of the forces operating upon ocean bars, led the writer to give them but little weight, and to conclude that the best guide in such cases was to be found in a correct interpretation of the resultant hydrographic features as revealed by a study of comparative charts. This method is best applied to alluvial coasts where the material, being more or less mobile or plastic, is moulded by the ever-varying forces of winds, waves, tides and currents, the resultant effect of which is manifested by the form, size and location of the adjacent and submerged shoals, dunes and other features; in short, it is a deduction of cause from effect and is characterized by certain typical forms and directions of movement, so that a comprehensive as well as detailed study of the records covering an extended period of time are important factors in solving problems of this class.

The writer, therefore, gave considerable time to the investigation of the literature and charts pertaining to harbor inlets, both at home and abroad, making personal visits to the more important works, and was surprised to find how small a percentage were designed with a view to utilize the natural forces for deepening and maintenance. He prepared several papers discussing and classifying these problems, which have appeared in various scientific periodicals.*

* "Harbor Studies." *Proceedings*, Engineers' Club of Philadelphia, Jan. 16th, 1886.

"The New York Entrance." *Proceedings*, Engineers' Club of Philadelphia, Feb. 6th, 1886.

"The Delaware Breakwater." *Proceedings*, Engineers' Club of Philadelphia, Feb. 20th, 1886.

"Problem of the New York Entrance." *Proceedings*, Am. Assoc. for Advancement of Science, Aug. 20th, 1886.

"Improvement of Tidal Rivers." *Journal* of the Franklin Inst., Sept., 1887.

"Improvement of the Port of Philadelphia." *Journal* of the Franklin Inst., Oct., 1887.

"The Physical Phenomena of Harbor Entrances: Their Causes and Remedies." *Proceedings*, Amer. Phil. Soc., 1888. (This paper received the award of the Magellanic Premium, Dec. 16th, 1887.)

"Discussion of the Dynamic Action of the Ocean in Building Bars, etc., being a reply to Report of the Board of Engineers, U. S. E." *Proceedings*, Amer. Phil. Soc., March, 1889.

(*This list continued in the foot-note to page 498.*)

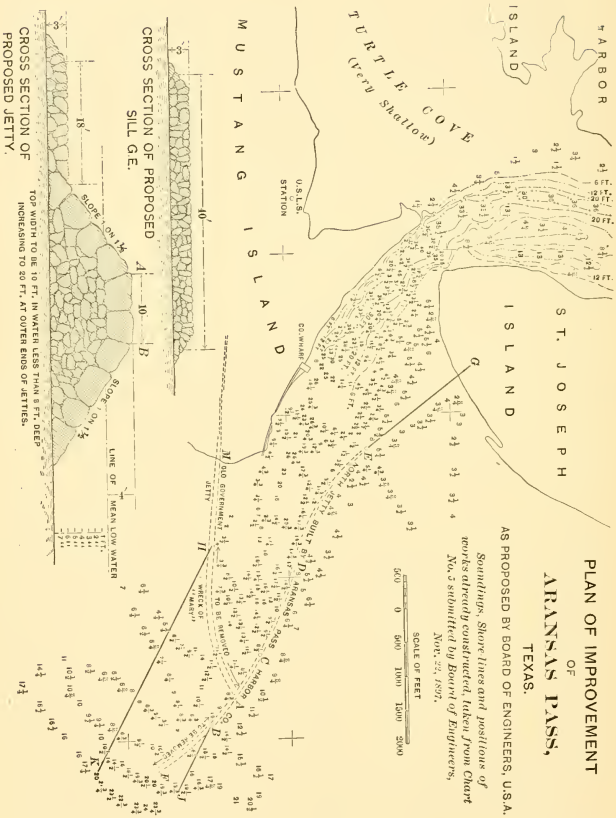

PLAN OF IMPROVEMENT
OF
ARANSAS PASS,
TEXAS.
AS PROPOSED BY BOARD OF ENGINEERS, U.S.A.
Soundings, Shore lines and positions of works already constructed, taken from Chart No. 5 submitted by Board of Engineers, Nov. 22, 1897.
SCALE OF FEET
500 0 500 1000 1500 2000
ARBOR
ISLAND
ST. JOSEPH
ISLAND
TURTLE COVE
(Very Shallow)
MUSTANG ISLAND
U.S.L.S. STATION
CO. WHARF
6 FT.
12 FT.
20 FT.
20 FT.
12 FT.
BLD. GOVERNMENT JETTY
WRECK OF "MARY"
BUILT BY ARANSAS PASS HARBOR CO.
CROSS SECTION OF PROPOSED SILL G.E.
CROSS SECTION OF PROPOSED JETTY.
SLOPE 1 ON 1½
SLOPE 1 ON 1½
LINE OF MEAN LOW WATER
TOP WIDTH TO BE 10 FT. IN WATER LESS THAN 8 FT. DEEP INCREASING TO 20 FT. AT OUTER ENDS OF JETTIES.

FIG. 1.

ARANSAS PASS, TEXAS.
SHOWING THE CONDITION OF CHANNEL CREATED BY THE UNFINISHED REACTION BREAKWATER.
MUSTANG ISLAND
SUBMERGED GOVERNMENT JETTY
WRECK OF "MARY"
GAPS
INCOMPLETE
SCALE OF FEET
0
600
1200
1800
2400
L. M. HAUPT, } CONSULTING ENGINEERS.
H. C. RIPLEY, }
REDUCED FROM ORIGINAL HYDROGRAPHIC SHEET, No. 2374
U. S. COAST AND GEODETIC SURVEY, FEBRUARY, 1899.
B. OLD JETTY BREACHED BY DYNAMITE IN 1896. SOUNDINGS IN FEET.

FIG. 2.

The result of these investigations led to the following conclusions:

(1) That in many places there were cyclic movements of the bars and channels produced by the resultant of the external littoral forces which were the bar-building agents, and which predominated over the internal or ebb currents which were the bar-destroying elements.

(2) That maritime engineers, in their efforts to improve purely tidal entrances, had greatly reduced the tidal energy by constructing two parallel or convergent jetties from the shore line to the bar, thus obstructing the free ingress of the tide upon which they were dependent for both volume and velocity at ebb, and that such construction confined the movements at all stages of both ebb and flood to the same path across the bar, thus producing no changes of equilibrium in favor of the ebb and requiring constant expense for bar removal and maintenance between jetties by dredging.

(3) That in the construction of jetties, where there is a resultant littoral drift, the wrong one is sometimes built first, thus arresting the drift and causing the bar to advance seaward (in one case nearly 3 miles), thus greatly increasing the cost.

(4) That no attempt had been made to utilize the reaction principle to effect a local scour across an ocean bar similar to that existing in all concave bends of streams.

The writer, therefore, believed that, by a radical departure from the existing practice, there were numerous localities where a single structure of proper form and position might be made to utilize all the available forces of Nature to assist in creating a better channel across ocean bars and fulfill all the requirements as set forth in the first portion of this paper, but as no precedent existed and he must needs establish the correctness of the theory, applications were made in January, 1888, to the Board of Engineers, U. S. A., for a report on his plans, followed by requests to the Chiefs of Engineers, under dates of June 30th, July 30th and September 14th, 1888, for permission to make a demonstration, at a site to be agreed upon. These requests were never even acknowledged, although the letters were doubtless received, since they were not returned, while the Board made an adverse report to the Chief of Engineers on March 16th, 1888, concluding with these words: The Board finds that Professor Haupt's views "are purely

"Fire Island Inlet: Is it Moving East or West?" *Journal* of the Franklin Inst., April, 1889.

"Report on the Harbor of Wilmington, Del." March, 1888.

"Jetties for Improving Estuaries." *Journal* of the Franklin Inst., April, 1888.

"Harbor Bar Improvements." *Journal* of the Franklin Inst., July, 1889.

"Galveston Harbor Problem." *Journal* of the Franklin Inst., Oct., 1891.

"How to Obtain Deep Water on the Gulf." Pub. Doc. S. 1583, 51st Congress, 1st Session, Jan., 1891.

"Plans for Improvement of Entrance of Rio Grande do Sul, Brazil," 1888.

"Littoral Movement of the New Jersey Coast, with Remarks on Beach Protection and Jetty Reaction." *Transactions*, Am. Soc. C. E., Sept., 1890.

(Most of the papers cited in this list will be found in the library of the Society.)

theoretical, are unconfirmed by experience and contain nothing not already well known, which has a useful application in the improvement of our harbors."

The writer only learned of the existence of this report by accident at a later date, when it was quoted against his plans. He then discussed the report, forwarding copies thereof to the members of the Board and the then Chief of Engineers, but received no reply. It was therefore impossible to "confirm the theory by experience," and there was no alternative but to await a favorable opportunity, which did not arrive until 1895, when, everything else having failed at Aransas Pass, where over $500 000 had been spent by the Government in futile efforts to secure 20 ft. of water, a private party finally concluded to build half of the reaction breakwater on a predicted depth of 15 ft. for this part of the work. This depth has now been exceeded, but only after a breach was made in the obstructing Government jetty, officially reported to have disappeared prior to the beginning of the work. The partially controlled currents have actually removed about 400 000 cu. yds. of compact sand from the bar without the aid of a dredge or other mechanical appliance, and have prevented the deposition in the channel of a much larger volume driven along the coast by the angular wave movements.

The theory, therefore, which has been successfully applied at this admittedly difficult and unpromising location, was to protect the bar crossing near its weakest point from the drifting sand by a resisting mass of rock so placed as to develop a continuous reaction across the bar by the ebb currents, preventing their expanding over the entire external sector and conserving the energy of the effluent stream until it reached the deep water of the Gulf.

As means to this end the breakwater is detached from the shore to permit the tidal compartments of the inner bays to be filled freely by each flood; it is also composed of curves whose radii and centers are adjusted to the site in such manner as to cause deposits on the outer side of the structure, thus re-enforcing it, and scour on the inner side, where an excess of foundation material revets the slope, and a peculiar order of construction is adopted, whereby the scour is assisted by gravity and the advance of the bar seaward is prevented.

All these results appear to have been fully secured by the work thus far done, as is shown by the Coast Survey report of February, 1899. Modifications, however, are always to be expected in depths due to variations in the natural forces, but the progressive deepening created by the currents gives every assurance of their ability to maintain the channel they have already created and which would be still further deepened by the completion of the work as designed.

The modifications submitted to the last Congress by two members of the Board of Engineers in their report of December 17th, 1898, are

shown in Fig. 1, from which it appears that it was proposed to return, as nearly as might be, to the old-time method of two, nearly parallel, jetties and the closure of the beach entrance, supplemented by dredging, at a cost of over $1 500 000. This plan, if adopted, would destroy the effective energy of the currents and prevent the completion of the demonstration of the principles which have been applied thus far at this entrance with unprecedented success, and would further involve an annual expense for maintenance, which is not now required.

This proposition, however, was rejected by Congress, which, as stated, made an appropriation of $60 000 to remove that portion of the old Government jetty which crossed the channel and provided that no portion of the reaction breakwater should be disturbed. This work was recommended by the United States District Engineer, June 8th, 1899, and was authorized July 11th, 1899, but, up to the latter part of September, the work had not been advertised to be let.

As a part of the evolution of the improvement of ocean bars and in confirmation of the theory, the writer desires to add that independent efforts were being made by others in the same direction and with substantially the same conclusions as to the primary importance of arresting the littoral drift and of concentrating the scouring force by a single curved breakwater instead of controlling the ebb movements by means of two jetties. As early as 1883, Mr. H. C. Ripley, then United States Assistant Engineer to Colonel S. M. Mansfield, stationed at Galveston, Tex., designed a concave, detached breakwater for that entrance, to be placed on the windward side of the channel, which was approved by Colonel Mansfield and recommended to the Chief of Engineers for adoption, in a letter dated March 19th, 1884. This design is shown in the Report of the Chief of Engineers for 1884, opposite page 1300, and designated: "Site of North Jetty, proposed by Colonel Mansfield."

This coincidence, in connection with Mr. Wisner's work, both at the South Pass and at the mouth of the Brazos, led to the formation of the Board of Consulting Engineers for Aransas Pass by Mr. Brewster Cameron, which consisted of H. C. Ripley, George Y. Wisner and the writer. Their predictions have been fully verified by the results secured by natural scour after the partial removal of the unexpected obstructions.

A propos of the severe floods in the basin of the Brazos River, Texas, in July, it was expected that the larger volume discharging with greater velocity over the bar at its mouth between parallel jetties would materially improve the depth. On the contrary, the soundings showed but 15 ft. in several places, thus demonstrating the relative superiority of the principle of reaction by a single jetty, as applied at Aransas, to that of concentration by twin jetties at the Brazos River bar.

DISCUSSION.

E. L. Corthell, M. Am. Soc. C. E. (by letter).—The writer hesitates to discuss this paper for the reason that it deals with an uncompleted work, which is so exceptional, as to the theory upon which it is based, that the paper describing it seems to be prematurely issued and unsatisfactory for discussion. Mr. Corthell.

The writer desired sincerely that the means should be furnished to complete the work according to the plans of its designers, in order that it might be demonstrated whether the peculiar theory upon which they are based is correct; for the cost will be only about one-third of the usual cost of obtaining deep water over sea bars.

The author, however, has had the boldness to attack the methods generally adopted hitherto, and he has advanced certain theories and made certain claims for them, and in so doing has necessarily criticised plans based on other and opposed theories. It is, therefore, necessary to consider the paper carefully, which, however, the present engagements of the writer do not permit him to do; he can only hastily give a *résumé* of the subject.

The terms used and the words coined by the author are not entirely understood by the writer, and he may not be correct in translating them into common language.

It is not quite clear what the word "reaction" means, or such an expression as this, "the curved form of the breakwater is such as to develop the potential energy of the ebb currents, rendering them kinetic." It is easily understood that a current impinging against the concave bank of a river scours out the bed near the bank, but just how this can exist in the case of a convex bank is not so easy to understand; for it must be remembered that one of the essential features of this work is its "S"-form—a "compound and reverse curve."

It might be expected in a river that a current passing from one concave bank to another concave bank, as is the case at the Aransas channel, would necessarily make what river men call a "crossing," and that on this crossing, or reversion point of the current, there would be a slackening of the velocity and a deposit of sediment. There seems to be such action going on at Aransas and a shoaling at this reversion, which no doubt will always exist and will give a limited depth at that point unless it is deepened artificially.

The entirely new idea that "it is not velocity * * * that constitutes the working force to produce scour, but reaction" is so entirely opposed to all accepted ideas of river and harbor engineers and their experience, that it is difficult to understand.

The writer, in the course of the last 25 years, has had occasion,

Mr. Corthell. not only to build works, but to examine many personally, not only in this, but in other countries, as well as to conduct an extensive correspondence with engineers engaged on similar works, and he must say that the views of the author are not, he believes, in accord with successful practice. The writer will not say that a single-legged, worm jetty would not, in some situations, obtain the desired results, but to adopt it as a general principle is, he believes, unsound.

The author refers, for confirmation of his views, to the Dublin harbor works, which the writer has examined, but has found so entirely different in conditions and plans from those at Aransas that he does not see the force of the comparison. The tidal opening at the land end of the North Wall at Dublin is only 600 ft. long, and the dike itself is 9000 ft. long. The tides in Dublin Bay not only rise 13 ft. as against a little over 1 ft. at Aransas, but this great range of tide—which, by the way, is semi-daily, instead of daily, as at Aransas—and the local configuration of the shores cause peculiar and opposing currents, which this opening and the direction and extent of the "North Wall" took advantage of; and—which constitutes the greatest difference—there is a solid wall 16 000 ft. long on the opposite side of the channel connected with the main land, all above water.

It is to be noted, further, that the channel, formed so successfully by scouring action through the bar, is at the narrowest place between the two jetties, and that where the "North Wall" flares away from the "South Wall" and leaves a very wide opening, the entire distance has to be dredged continually to make and maintain the depth. In fact, from the bridge at Carlisle Street, in the port, to within 1 000 ft. of the sea end of the "South Wall," which is the guiding and longest jetty, the improvement in the channel is due solely to dredging, the effect of the scour being inappreciable.

One purpose of the Aransas breakwater is to "arrest the littoral drift," and for this purpose it is built on the "windward" side of the channel. The author presents no data to show that the "windward" side is the north and east side. The writer's recollection is that the preponderance of winds is from the south and west, and that the winds and the currents are up the coast during most of the year. However, that is a matter of fact to be ascertained from the anemometer records at Aransas or Galveston. As to currents in the Gulf of Mexico, the writer believes the following to be generally the conditions:

The current enters through the Yucatan Channel, and goes out through the Florida Channel—so far, at least, as surface currents are concerned. The former moves up the coast, hugging it in the Bay of Campeachy, past the mouth of the Coatzacoalcos River at the Isthmus of Tehuantepec, past Vera Cruz, and then begins to spread out into the great area of the Gulf. Nearing the mouth of the Mississippi,

but still some distance off, it gathers itself together for its passage through the Florida Straits, its velocity gradually becoming greater. All along this part of the coast there is a reactionary or eddy current which moves in the opposite direction, and which is felt as far west as Galveston and the mouth of the Brazos, though with much diminished force. At Aransas there is no appearance of the Gulf Stream for probably 150 miles off shore; it is opposite the widest expansion and the slackest current of the Gulf Stream. The winds cause the currents, and but little reliance can be placed on the westward drift. For this reason the opening which is left between the jetty and the land is of only theoretical advantage, and the writer believes it to be a positive, practical disadvantage. This was his contention in reference to the old plans at Galveston, and, as a lesson from these works, the following is abstracted from his paper on the South Pass Jetties:* Mr. Corthell.

The flood volume of the river, and the full volume of the tides, are the powers which the engineer should call to his aid. Lateral outlets near the land should not be provided. Ostensibly, they are for the purpose of letting the tides into the bays, but it is through them at the ebb that they seek to find the lower level of the sea, instead of taking the longer and more difficult course along the inside of the works, where there is a pronounced frictional resistance to current movement.

This important feature of harbor improvement was explained so clearly by the late James B. Eads, M. Am. Soc. C. E., in his discussion of the writer's paper that it will be useful to refer to it in order to set ourselves right on this point.

The writer has made a special study of most of the river mouths and harbor entrances on the North, Baltic and Zuyder Seas—there are over 25 of them—and he knows of no instance where there are not two jetties contracting the channel-way and concentrating the fluvial or tidal flow. These parallel jetties are not always equal in length, but there are always two of them; and while in the nearly tideless seas of the Baltic and the Zuyder Zee dredging has to be done to some extent, there is no doubt whatever that the two jetties are the cause of the successful maintenance of the deep channels which have been maintained there for many years, the depths of which are gradually increased to accommodate the increasing drafts of vessels.

Further, before quoting Mr. Eads' discussion, the writer desires to state some facts, in reference to the Tampico harbor works, which may have a bearing on the "reaction" theory of the author. They will at least furnish a remarkable proof of the correctness of Mr. Eads' views, expressed seven years before their occurrence. The details of the history of the Tampico works will be found in the *Proceedings* of the Institution of Civil Engineers.†

* *Transactions*, Am. Soc. C. E., Vol. xiii, p. 326.

† *Minutes of Proceedings*, Inst. C. E., Vol. cxxv, 1895-96, pp. 243-262.

Mr. Corthell. The Pánuco River, which flows into the Gulf of Mexico at Tampico, is one of the largest in Mexico. The area of its water-shed is about 36 000 square miles. The periodicity of its floods is very variable. Sometimes there is a period of 3 to 5 years between them. The works at the mouth of the river consist of two parallel rectilinear jetties, 1 000 ft. between center lines. They are built of rough stone, with a foundation and hearting of mattress work. The depth on the bar about one mile from the shore was about 8 ft.

The works were pushed to completion with great rapidity. They were completed in December, 1892. Two or three small rises, and the tidal flow had deepened the channel to about 17 ft.

In July, 1893, a heavy rise came. It found its usual outlet partially dammed—walls on the sides and a bar between. It raised the surface of the water at least 2 ft. at Tampico, 7 miles from the Gulf; but the force brought to play upon the bar was tremendous. Slope observations made during the rise—which lasted from July 12th to August 1st—showed that the fall of the surface at the sea end of the jetties varied from about 0.5 to 2.75 ft. per mile. The velocity was over 8 miles per hour. The total amount of material scoured out of the bar and discharged into the sea in that flood was 1 201 985 cu. yds., in addition to about 7 500 000 cu. yds. of suspended matter.

The "reaction" upon this bar by these forces consisted simply in moving a great volume of water down a steep slope; it acted vigorously upon the bed of the stream and swept it out to sea.

Another great flood, in 1895, accomplished the same results, but it, finding a deep channel, spent its force in widening and deepening it the entire distance between Tampico and the mouth of the river, and restoring the sections which had become contracted there by the deposit of sediment in the floods which had occurred during the construction of the works.

With these facts—and similar ones from other works might be mentioned—let the reasons for such satisfactory results, given in discussing the writer's paper on the South Pass Jetties and the Galveston harbor plans, be stated by Mr. Eads:*

"The Jetty System Explained.

"It is known to all engineers familiar with harbor improvements, that the jetty system is a method of deepening and maintaining a channel across a shoal by such works as shall compel the water flowing over the shoal to pass through a narrower channel; but, as this paper will be read by some who are not familiar with harbor improvements, I will be pardoned for stating the following elementary principles which control such problems:

"*First.*—A current is caused by the fall of water from a higher to a lower level, which fall is indicated by the slope or inclination of the surface of the water.

* *Transactions*, Am. Soc. C. E., Vol. xv, p. 284, *et seq.*

Mr. Corthell.

"*Second.*—Friction of the bed over which the water flows is the chief force opposed to the current.

"*Third.*—The velocity of the current will be increased by either increasing the slope of surface or by increasing the volume, or by lessening the friction.

"*Fourth.*—The friction increases as the width of the bed increases; that is to say, if the bed of the channel be twice as wide, the friction will be doubled; if its width be reduced one-half, the friction will be reduced one-half.

"*Fifth.*—The power of water to transport sand increases with the square of the velocity.

"As every grain of sand that lies upon the shoal is brought by the current and left there, because it could take it no further, it follows that we cannot hope to deepen the channel, except by increasing the current. This can be done in three ways, but in this case it is impossible to do it *at first*, except by increasing the slope of surface. After the current is *increased*, deepening commences, and as the contracted channel enlarges, the slope is gradually reduced to its former inclination. When this occurs the old rate of current would return, but the other two elements, namely, *increase of volume* and *diminished friction*, then continue the abnormal acceleration. As the deepening goes on a *reduction of slope* follows, and this finally reduces the velocity so much that deepening ceases. The slope results from the difference of the heights of the water in the Bay and the Gulf. *To facilitate* the flow from one to the other, by *leaving openings through the jetties*, tends to keep these heights nearer alike and the resulting slope between them must then *be less*, not greater. This is precisely what we do not want *at first*. * * * * * * * *

"Outlets Through the Jetties.

"As the reader may not at once perceive the mistake of leaving these openings to facilitate the flow into the Bay, I will explain that, in a state of nature, the immense frictional surface over which the flood water has to pass in filling the bay retards it so much, that the tidal rise in the Bay at Galveston is probably 10% less than it is outside of the bar. On the other hand, the discharge of the ebb is retarded by the same cause, making the total rise and fall just inside of the harbor probably 20% less than it is at the same depth outside of the bar.

"The bottom of the channel is composed of separate grains of sand, without the slightest cohesion or bond. They were brought here by currents and left, because those currents, from some cause or other, became too feeble to carry them further. Now, the reader must remember, first, that a current depends upon *the slope of surface* of the water (not upon the slope of the bottom), and second, that the excavating or transporting power of the water depends upon the *velocity of the current*. This excavating power or *velocity of the current must be made greater* than it now is over the bar, or the channel cannot be permanently deepened by any artificial structures known to the science of engineering. And yet no engineer can increase this velocity *permanently*, as will soon appear. But the velocity *must*, nevertheless, be increased, or these masses of sand composing the bar cannot be *permanently* removed How can the velocity of a current be increased? In three ways. First, by increasing the *slope of surface*, which is the cause (or rather the exemplar of the cause) of the current. Second, by lessening the frictional resistance of the bed. Third, by increasing the volume

Mr. Corthell. or mass of the water. We cannot avail of these last two methods *at first*, but after we shall have *once increased* the velocity by permanent works, these two last means will then ultimately come to our aid. *At first* we have absolutely no power to increase the velocity of current but by *increasing the slope of surface.* These slopes are alternately towards the sea and from the sea, accordingly as the tide fills and empties the bay. They are by no means steep, as the average maximum tides are but a little over a foot high. The force of the present currents resulting from these low slopes cannot overcome the force of gravity in the grains of sand composing the bed of the channel. With the present slopes, volume and frictional resistance, the equilibrium between the *velocity* or transporting force of the current and the force of gravity in the sand is reached when the depth of 13 ft. occurs on the bar. If we should suddenly deepen the channel by dredging, it would be larger than Nature demands under the present conditions. The bay would then fill more easily, the slopes would be lower, the velocity less, and soon after the dredging ceased, the enfeebled current would drop its burden of sand in the enlargement, and restore the present depth. On the other hand, if we could suddenly drop a million tons of sand in the channel, and reduce the depth a few feet, what would occur? The bay would not fill so rapidly, and steeper slopes would occur in consequence. These would create greater current velocities, and the removal of the obstruction would be effected in a little while; and, with the recovery of the 13 ft., the bay would fill more easily, the slopes would diminish, and the former velocities of current would be restored. The slope results from the difference of level between the surfaces of the Gulf and the Bay. Any one can see that the more freely the water can flow from one to the other, the less difference will there be between the heights of their respective surfaces, and, consequently, the less will be the slope of surface through the channel between them, and the less will be the current velocity. This is certainly the very opposite of what *we must have* to produce deepening. We must have more rapid currents, and nothing but steeper slopes will produce them. Hence it is evident that *large openings*, and a freer communication with the bay through them, and over low, submerged jetties, are precisely what we do not want.

"Let us suppose that jetties were, at mean tide, suddenly placed across the bar, 3 000 ft. apart, and extended from 30 ft. depth inside to 30 ft. depth outside, and were so high and tight that all the water which passes in or out of the bay is compelled to pass through them. When the tide rises it will be impossible for the bay to fill as fast as before. The tide outside rising only to its normal height will produce a steeper slope through the jetty channel than existed in that locality before, because the surface of the bay must be lower, as it will have been deprived of a large part of its normal volume of the flood. This steeper slope will produce a more rapid current into the bay, over sands which are only quiescent under the present velocity. If this slope increases that velocity from 3 to 4 miles per hour, which is not an unreasonable supposition, the transporting power of the water will be increased from 9 units to 16 units. Wherever through this jetty channel the depth is least, there will the current be most rapid. Hence the inflow through it will have the effect of cutting down the high parts of the bottom first, and transporting these sands into the deeper places. In a few hours this whole process is reversed, and a steeper slope than before will exist in the opposite direction, and the sands of this contracted channel will be swept out to the sea, and will there be

distributed by littoral currents or sea waves, out of the way of the out- Mr. Corthell.
flow from the jetties. Of course, the greatest increase of slope will only occur at first, and at the highest and lowest periods of the tide; but *as the channel deepens the volume increases, and the ratio of friction to volume decreases*, hence lower slopes will then produce equally rapid currents. Therefore the bay will ultimately fill more and more quickly, and these abnormal currents will be thus toned down until the deepening ceases and a new condition of stable equilibrium, or what engineers call "a new regimen," is established between the maximum velocity of the current and the force of gravity in the sand. When this is reached it will be found that the tidal movement in the harbor conforms more nearly to the rise and fall in the Gulf, both in height and time, and that the slopes are *lower* than at present. Because the deepening of the channel will continue until the bay will again receive all of the water due to the difference in the tidal movement, and to the normal slopes of surface occurring between the alternating levels of bay and sea through this deep channel, and as the friction which now retards the entry and exit of the water will be greatly reduced through it, a correspondingly increased quantity of water will enter the bay.

"Any excessive increase of tide caused by winds will have the effect of permanently increasing the depth of this jetty channel, and of making it more capable of discharging such excessive quantities of water again. By such abnormal causes the channel will be so deepened that the currents from ordinary tides will not be sufficient to disturb its bottom. The greatest work of excavation is done when the velocities are greatest. The channel with high jetties extending out to deep water would get no accessions of sand, and the depth would be permanent. It must be apparent, from this explanation, that the object sought by these officers, namely, *facilitating the inflow of the tide* into the Bays at Galveston, Charleston, Sabine Pass, etc., by leaving openings through the jetties near the shore, is precisely the way *not to do* what they want to accomplish. Besides losing through them the most important agent for deepening the channel—the ebb tide—they will admit into the channel all of the sand which the sea waves can bring. *Submerged jetties* simply aggravate the evils caused by these openings. They really extend the openings over the entire length of the jetty. They prevent, by increased friction, the rapid inflow to fill the bay; they permit the loss of the ebb tide, and thus lessen the force that should be applied to deepen the channel; and they allow the waves to transport sand over them into the channel. Openings and submerged jetties are therefore an inexcusable violation of the very essence of the jetty system."

There are some other serious objections to the single-jetty plan at Aransas. There is no concentration of the ebb-tide flow to make and maintain a deep and wide channel. There may be a narrow, curved channel along the concave side of the work, but between that and the outlet at Mustang Island there is always likely to be a troublesome shoal due to the reversion point of the current previously referred to. And beyond, at the sea end of the curved jetty, there is always likely to be a shoal bar for want of a strong issuing current to sweep it away. Again, the channel throughout its whole length along the curved breakwater is exposed to the sudden and serious influx of sand at any time by southerly storms.

Mr. Corthell. In the case of the South Pass and Tampico Jetties, the bars in the sea are swept by currents passing across the ends of the jetties. The winds and storms from the southeast at the former, and the "Northers" at the latter, raise the surface of the water in the angle formed by the East and North Jetties and the shore line and produce a strong current around the windward jetty and across the fore shore of the sea bar, wearing it away and thus retarding its growth. There is no such force to rely upon at Aransas, for the wide opening between the land and the breakwater will prevent it.

This discussion is not upon a subject unfamiliar to the writer. He was the chief engineer of the project for two years during its inception, had charge of the matter when the charter was obtained from Congress in 1890, and made plans and estimates of the works, which consisted of two parallel, rectilinear jetties. He is confident, from his experience elsewhere and knowledge of the conditions at Aransas, that his plans, if carried out, would have given immediately a channel 26 ft. deep and at least 200 ft. wide at that depth, which could have been maintained easily by keeping up the works, and without resort to dredging.

It remains to call attention briefly to the closing paragraph of the author's appendix, in which, after making an erroneous statement about the effects of the recent floods at the Brazos, he says:

"Thus demonstrating the relative superiority of the principle of reaction by a single jetty, as applied at Aransas, to that of concentration by twin jetties at the Brazos River bar."

Let the writer say, first, there is no sort of comparison between the conditions at the two places. At Aransas there is simply a clean, salt-water, tidal connection between the Gulf of Mexico and a group of bays—Aransas Bay with 81½ square miles connected with Misquete, St. Charles, Copano and Corpus Christi Bays, which together have an area of 273½ square miles. At the Brazos, there is a great, turbulent, muddy river in times of flood, 1 000 miles long, flowing through a clayey country, and in floods vomiting its engorgement out upon the Gulf; with no tidal reservoir whatever, except what exists in a narrow river not over 400 ft. wide.

At Aransas, the normal depth on the bar was about 9 ft. At the Brazos, when the works were begun, there was a straggling channel of 5 ft., which had to be crossed by the East Jetty. The depth on the bar between the jetty lines was only 1.5 ft. Still, these works, though never completed or consolidated, due to financial reverses by the financiers of the project and for reasons outside of this project, deepened the bar and made a channel straight to sea, which, in 1896, had a depth of over 20 ft. at average flood tide, the plane of reference used.

The author states:

"Apropos of the severe floods in the basin of the Brazos River, Texas, in July, it was expected that the larger volume discharging

with greater velocity over the bar at its mouth between parallel jetties would materially improve the depth. On the contrary, the soundings show but 15 ft. in several places." Mr. Corthell.

The author was misinformed; for the writer communicated with the works and ascertained the following from the engineer and superintendent of the railroad and terminal:

"October 3d, 1899, Velasco, Texas.

"Letter just received. First part rise deepened channel immediately outside jetties to 23 ft., mean low water. Latter part shoaled same place to 15 ft. in one small place, but deepened again to 16.5 ft. within two weeks; now 18 ft. Very marked improvement in river channel from Velasco to and through jetties. Never such depth water before; no harm to jetties. Bar entirely outside jetties. Now channel deeper and wider in front of jetties than ever.

"E. D. Dorchester."

The writer also communicated with Mr. William H. Coolidge, the attorney of the Reorganization Committee, in Boston, which has general charge of the property, and the following was received from him:

"My instructions from Velasco have been to the effect that almost without exception the flood removed bars, etc., so that there was a clear entrance drawing at least 18 ft., and with a very slight amount of work the depth would be 22 or 23 ft. With two or three very slight exceptions, the water was more than 20 ft. deep. The Government having taken charge of this property, I am told that a very few days of dredging will allow vessels to enter drawing 23 or 24 feet."

The writer's original report, made after one had been made by the Government Engineer to the effect that 10 ft. as a maximum might be maintained by jetties with a maintained channel of 6 ft. only; and that the $140 000, already expended by the Government without result, should not be supplemented by any additional amount, gave the opinion that a channel 20 ft. deep could be obtained and maintained by properly constructed jetties. He still believes this can be done, notwithstanding the unfortunate financial history of the work, the delays in construction and the inability of the company to complete the works at the time.

Henry L. Marindin, M. Am. Soc. C. E. (by letter).—The method of improvement designed by Professor Haupt for Aransas Pass presents three prominent features, the most important of which, in the writer's estimation, is the "reaction principle" as shown in the curvature given to the breakwater. Mr. Marindin.

The principle is based upon the well-known effect produced by flowing water in the concave bend of a stream where the impingement of the current causes greater depth than in any other part of the cross-section, and, indeed, greater than in the straight reach.

The frequency with which running water flows in curves would seem to amount to a law of flowing streams, and this is nowhere illustrated better than in the case where the stream is free to carve out its own bed, as in the Lower Mississippi; there the regimen lies in cur-

Mr. Marindin. vature, and it is only when this curvature becomes excessive, and offers too much resistance to the flow, that during some period of flood stage the water takes a straight course—in the line of least resistance—across the more or less narrow neck of land separating it from the reach below—making what is known there as a "cut-off;" but this remedy is as much a violation of the regimen in one direction as the excessive curvature of the bend which caused it was in the opposite direction. The stream almost immediately proceeds to form another bend.

That it is not the mere velocity of current which induces erosion of the bottom of a channel must be acknowledged by witnessing so many instances where, in one case, a stronger current fails to produce erosion of the bottom, and in the other case, with the bottom composed of—to all appearance—similar material, a weaker current maintains greater corresponding depths.

The New York Narrows is offered by Professor Haupt as an illustration in point, and is an apt one so far as that a great depth of 100 ft. seems to be maintained by the comparatively low mean velocity of 2 ft. per second. There are instances where greater mean velocities fail to maintain 10% of the depth found in the Narrows.

The explanation offered by Professor Haupt for the great depth in New York Narrows is that a resultant force is produced by the flood stream flowing up on the bottom for eleven hours out of the twelve, indicating a huge vertical eddy. That may be the true explanation, but, in that connection, the writer may be permitted to ask if he has not misunderstood the report from which the statement is taken? Seven hours out of twelve would be more in accord with what has been observed in other parts of the harbor.

The flood current has been observed to flow up on the bottom, at the mouth of the Hudson, for $2\frac{1}{2}$ hours before the ebb current had ceased to flow at the surface.

It has also been held that if both ebb and flood currents could be made to follow the same path or channel, although in opposite directions, the problem of improving the depth of outlets would be near a solution. This is seemingly the condition at New York Narrows.

The configuration of the shores at Aransas Pass seemed fitted for a "compound and reverse-curve breakwater." The position of the concave shore of Mustang Island made the convex part of the breakwater opposite to it possible, or, perhaps, better fitted than a straight jetty would have been, and, as the thalweg is found close to the shore, with ample depth, it only remained for this part of the breakwater to arrest any encroachment upon the channel from the north and east. Farther down, the reaction principle becomes operative until the deeper water is reached, beyond the bar.

It is not clear to the writer's mind wherein Professor Haupt has

provided for meeting the requirement stated in his seventh specification, viz.: Mr. Marindin.

"It (the breakwater) should not promote the growth of the bar seaward by the deposit of material removed from the channel on the outer slope of the bar."

If material is being moved by storm waves, some of it will find its way past the outer end of the breakwater, and any material held in suspension or rolled along by the ebb on the bottom of the channel will be deposited in front of the outlet, no matter what form shall have been given to the end of the breakwater.

The writer does not know how much or how little sediment is brought down by the ebb out of the Pass, but if any amount is being carried, and a bar tends to form in front of the present curved end of the breakwater, an extension in continuation of the curve could not be continued for any great distance, for obvious reasons; and a reverse curve would be detrimental to the reaction theory, without transferring the reaction principle over a "crossing" to another concave breakwater located on the south side of the channel. Such a crossing means the formation of a middle ground, with decreased channel depth.

The second important feature in this improvement lies in having provided a wide gap at the shore end of the breakwater for the free passage of the flood tide, to fill the tidal reservoirs above the Pass, at a point which is not occupied by any considerable ebb stream, the strength of the ebb occupying, as it does, the concave shore of Mustang Island.

It is evident that where the working power of the ebb depends largely upon the drain of flood-tide waters, works of improvement should be so designed and constructed as to offer the least obstacle to the free access of the flood tide; under these conditions the building of continuous parallel or convergent jetties would be a mistake.

The third prominent feature is the location of the single curved breakwater on the side of the Pass from which may be expected the most violent movement of material, or that due to a littoral drift. The determination of this location should be based upon accurate and reliable information, previously obtained, as to the direction and strength of flood, the direction and volume of drifting sands or other material caused by prevailing winds and storm waves or by any littoral current. Professor Haupt disclaims having sought or secured any of these data, preferring the study of the topographic and hydrographic features shown on published charts, supplemented, as it must be, by the intuition of the engineer. This is well, if success follows it, but the writer is sure that the majority of engineers would prefer to have numerical data before them to mix with all the intuitive knowledge they may possess.

An analysis of the systems of parallel or convergent jetties and of Professor Haupt's system, as applied to Aransas Pass, shows them to

Mr. Marindin. be radically different. The application of the single "Reaction Breakwater" seems, from the results noted in the paper, to have been particularly successful in a very short time, notwithstanding the existence of an old rock jetty directly in its path and removed only in part. But the same system applied to South Pass or to South West Pass would prove a failure. In both of these cases the conservation of the energy of the current which obtains between their parallel shores and its maintenance as far as the bar, is necessary, not only to erode the bar, but to convey the millions of tons of solid matter, held in suspension and rolled along the bottom, far out into the deep water in front of the delta, where the rebuilding of the bar would be a very slow process, retarded by the action of any littoral current existing there. Parallel and continuous jetties are necessary to secure these results, and prevent the spilling of the currents laterally.

The same condition confronted the engineer at the mouth of the Brazos, and it may be said, parenthetically, that the present jetties in their incomplete state have secured within their limits as great depths as are found in the river a mile or two above the month.

The writer has endeavored to show in the foregoing remarks that, notwithstanding the success which has attended the single jetty, parallel jetties are justifiable and necessary under certain conditions.

Mr. Wisner. GEORGE Y. WISNER, M. Am. Soc. C. E. (by letter).—The peculiar nature of the reports made by the Board of Engineers on Aransas Pass, in November, 1897, and December, 1898, together with the fact that the expenditure of large appropriations will soon be required for the improvement of other harbors where the conditions are similar to those at Aransas Pass, make the paper presented by Professor Haupt very apropos at the present time.

The harbors on the Gulf Coast are of two distinct types, each of which requires radically different treatment in order to secure satisfactory results. The Mississippi River, the Brazos River and the Pánuco River, at Tampico, are all heavy silt-bearing streams during times of high water, and, if not controlled by tight jetties so as to maintain the full velocity of flow until discharged across the bar into the littoral current outside, a deposit of silt will occur and produce shoals instead of deepening the channel as generally expected. The strong currents of these rivers at flood stages are alone to be depended upon to produce scour in the channels, and all arrangements designed for securing tidal effect are not only useless, but tend to decrease the velocity of the silt-laden current, and thereby cause shoals by depositing silt.

At the entrances of tidal harbors, reversed conditions exist, and the erosive force required to deepen the channels must be obtained by accumulating vast quantities of water in the tidal reservoir, during the

flood stages of the tide, and concentrating the discharge along the axis of the required channel lines during ebb tide. Mr. Wisner.

Unfortunately for the success of many of the improvements attempted, the designers of the structures conceived the idea that works which had produced beneficial results at other harbors must necessarily be the proper remedy to apply. The result has been that millions of dollars have been expended on jetties which have been of absolutely no use, except to form breakwaters for the protection of dredged channels. One of the most serious difficulties which the designers of such works have apparently feared was that such strong tidal currents would be generated that the structures would be in constant danger of being destroyed, and, to avoid such a catastrophe, the jetties have invariably been located from 2 000 to 7 000 ft. apart.

Generally speaking, no deepening of channels has resulted from such construction, and whatever deeper channels now exist at these harbors have been secured by subsequent dredging.

At Galveston, with jetties 7 000 ft. apart, the dredged channel across the bar is from 500 to 1 000 ft. wide, and at Sabine Pass, where the jetties are 2 000 ft. apart, the dredged channel is only about 200 ft. wide, and shows no indication of increasing depth or width from tidal action. In spite of these repeated failures, the 1897 Board of Engineers not only endorsed the 1888 project for jetties 2 000 ft. apart at Aransas Pass, but condemned the unfinished curved jetty as worthless to the Government, and as an obstacle to future improvements, when their own published charts show beyond question that the results predicted for the enterprise would be realized at once upon the removal of the old curved jetty, which the predecessors of the Board located, unfortunately, on the wrong side of the Pass.

In the report of December, 1898, the Board recommends that the outer 1 000 ft. of the curved breakwater be removed, and that an extension be built on a line about 25° to the north of the present location, making an angle with the direction of flow which, it is safe to say, the currents would never follow. With a channel from 15 to 23 ft. deep, already obtained by the incomplete works now in place, it is hard to conceive why such structures should be removed, and be replaced by others which have been proven to be failures at other ports, where the conditions were similar, especially as the estimated cost is three times that necessary to obtain a 20-ft. channel by the completion of the works built by the Aransas Pass Harbor Company.

While it is true, as stated by the author, that parallel jetties are not well adapted for securing tidal scour at the entrance of tidal harbors, it is equally true that a single curved jetty at the mouth of a silt-bearing river would not only fail to produce a good entrance channel, but would not maintain it if obtained by dredging or other methods. The case mentioned, where the flood in the Brazos River

Mr. Wisner. failed to deepen the channel, is a direct proof of the principle involved, and was the result expected by those familiar with the situation.

The jetties at the mouth of the Brazos River have never been completed so as to maintain a full velocity of flow until the silt-laden water is discharged into the deep water of the littoral current outside the bar, and, therefore, in times of heavy floods a portion of the sediment is dropped at the point where the river current is slackened. The experience at the mouth of the Mississippi and Brazos Rivers indicates clearly that single jetties, no matter what their shape, would not maintain navigable channels at the entrances of those harbors.

To every careful investigator of the physical conditions at the entrances of Gulf harbors, there is plenty of evidence of the kind of remedy which should be applied to secure any reasonable improvements of the channels. The littoral current is a controlling force, which is certain to cause either success or failure, according to whether properly utilized or not.

At the mouths of silt-bearing streams, the suspended matter must be carried entirely from in front of the entrance before being deposited by a change in the velocity of the river current, and the shore drift must be prevented from entering the channel; two conditions which necessitate the construction of jetties on each side of the entrance, to deep water on the outside of the bar. The action of the littoral current in flowing past any system of jetties is to pile up the sand drift in the angle of the shore and jetty on the windward side, and produce an eddy on the lee side of the entrance.

On the Gulf Coast, the resultant of the littoral current is from the northeast to the southwest, and in the case of sediment-bearing rivers the eddy builds up immense shoals to the westward of the entrance, and at tidal harbors causes a reverse current along the shore to the westward of the jetties, which in connection with wave action may cause great damage by erosion.

This shore current, caused by the eddy to the westward of the jetties, has in certain instances led observers to believe that the littoral current was from the west.

At Galveston, the erosion of the city front from this cause has been so extensive that suits have been started in the Court of Claims to recover compensation for the damages caused by the construction of the jetties at the harbor entrance.

If the curved jetty designed for the improvement of Galveston Harbor, in 1883, by H. C. Ripley, M. Am. Soc. C. E., had been adopted, it would unquestionably have produced a deep channel across the bar with a saving of several million dollars in the cost of the improvement, and would have shifted the eddy caused by the littoral current far enough eastward to have caused but little erosion along the city front.

Mr. Wisner.

The construction of jetties at Aransas Pass will produce an eddy effect similar to that developed by the structures at the entrance of Galveston Harbor, and, if two jetties are built out to deep water, the west end of Mustang Island will be eroded, whereas, if only a single curved jetty be constructed on the north side of the channel, the center of the eddy will be shifted eastward, and the reverse current will combine with the ebb currents at the outer end of the channel. A short spur will be a necessity at the west side of the Pass to prevent the reverse current from the littoral eddy carrying sand into the channel. This spur should be built above high water as far out as the wreck *Mary*, and will tend to limit the action of the eddy and diminish erosion on the front of Mustang Island.

One of the most interesting instances of parallel jetties producing the opposite effect to that expected is that at Cumberland Sound, Ga., where the accumulation of drift between jetties 3 900 ft. apart became such that, in 1895, the channel shifted across the south jetty, and it became necessary to remove a portion of that jetty, to prevent the port of Fernandina from being completely closed.

It became evident, in 1890, that such a result was likely to be obtained, and O. M. Carter, M. Am. Soc. C. E., Captain, Corps of Engineers, U. S. A., the local engineer in charge of the improvement, recommended that the project for parallel jetties be abandoned, and that a single curved jetty be constructed on the north side of the entrance, to shut out the sand drift from the north and guide the tidal flow across the bar on the line best adapted for maintaining a channel with natural forces.

The deadly wisdom of the engineer board, whose consideration of a few hours outweighs the conclusions of the resident engineer who gives years of study to the conditions, proved fatal to the project.

There has been $1 787 000 expended on the work, which has in no way improved the entrance to the harbor. A continuous appropriation was made by Congress in 1896, to complete this project, of which over $1 500 000 remains unexpended.

It will be interesting to note how this vast amount of money is to be expended, especially as the engineer boards which have considered the problem at various times have practically admitted that the improvement was a failure, but did not have the courage to "advise any radical change in the general location of the jetties as originally proposed."

The conditions at Cumberland Sound are practically similar to those on the Gulf Coast, except that the rise of the tide is about three times that at Aransas Pass; and there is no reason why a single jetty would not produce as beneficial results as at the latter place, where an incomplete structure has deepened the channel from 8 to 18 ft. in a few months.

Mr. Wisner. Particular attention is called to the fact that the breakwater at Aransas Pass was constructed under the plans of the consulting engineers, without causing the bar to be pushed seaward the slightest measurable amount, an achievement which has never been accomplished at any other port on the Gulf Coast.

The action of erosive and bar-forming currents has been quite fully observed by the engineers who have had the actual charge of harbor construction on the Gulf Coast, and, while the results have not been published in the official reports, many of the facts have been discussed in the *Transactions* of this Society, and it is hoped that this paper will make the record still more complete.

So far as the writer is able to judge, these observations have established quite fully the following conditions and principles pertaining to harbors on the Gulf Coast.

1. The littoral current, during a season when effective, flows from the northeast to the southwest.

2. The sand moved along the coast by the littoral current, when stopped by jetties extending into the Gulf, accumulates in the angle formed by the east jetty with the shore; and the sediment carried into the Gulf by silt-bearing rivers is carried to the westward of the harbors, forming shoals extending seaward from the entrance of the jetty channels.

3. The littoral current flowing past the outer ends of jetties or breakwaters creates an eddy west of the entrance, which, at the mouths of sediment-bearing streams, builds up shoals in the quiet water at the center of the eddy; and at the entrances of tidal harbors causes reverse currents, which, in connection with wave action, erode the shore west of the structures.

4. A single jetty will neither make nor maintain a navigable channel at the mouth of a silt-bearing river.

5. Parallel jetties at the entrances of tidal harbors will not create navigable channels, but will generally maintain such a channel after having been obtained by dredging.

6. Where strong littoral currents exist during the season when sand movement occurs, a single curved jetty, properly designed with reference to the volume and velocity of flow through the pass, will create a navigable channel without aid from dredging, but it is preferable in all cases to aid the tidal action by dredging, not only to hasten results, but to insure a proper alignment of the channel.

Mr. Symons. Thomas W. Symons, M. Am. Soc. C. E. (by letter).—The single jetty at Aransas Pass, or "reaction breakwater," as it is called by the author, is by no means a new thing in the engineering world, and, individually, it does not mark "a distinct advance in the resources of maritime engineers in the improvement of ocean bars," as stated by Professor Haupt. In fact, officers of the Corps of Engineers, U. S. A.,

have designed and have been building just such structures for years, and some of the most successful works of the Corps have been accomplished by adopting in the main the principles enunciated by Professor Haupt. Mr. Symons.

The writer believes sincerely that many harbor bars can be improved by a single jetty, or breakwater, if the name be preferred, to fully as great an extent, and in some cases to a much greater extent, than by parallel or converging jetties, and at enormously less cost, provided the single jetty be properly located to take advantage of all the natural forces at work. This belief is founded, not on theories only, but on actual experience in planning, locating, building and observing such works and their operation. The writer's experience on the Pacific Coast was so convincing as to this, that, in planning works for the improvement of Grays Harbor Bar, Wash., he designed a single jetty, and, Congress having made provision therefor, it is now being constructed under the supervision of Harry Taylor, M. Am. Soc. C. E., Captain, Corps of Engineers, U. S. A. This jetty has not only been described in official reports, but was the subject of a paper before this Society.* This Grays Harbor jetty is, in every essential respect, such a "reaction breakwater" as described in general terms by Professor Haupt.

The single jetty at Coos Bay, Ore., located and built under the writer's direction and supervision, 1890 to 1895, is a "reaction breakwater," and operates in practically the same manner as described by the author, in determining, deepening and maintaining a channel over the bar. This single jetty was located with a view to securing the maximum results in the shortest time and at the minimum of expense, and was eminently successful; a deep bar channel was developed and has been maintained in a permanent position ever since. A second jetty was planned to be built at Coos Bay, but as the first one has accomplished about all that was hoped from the two, it will probably be deemed unnecessary to build it. The great single jetty at the mouth of the Columbia River, which has accomplished so much for the commerce of Oregon, is a "reaction breakwater." So is the single jetty at San Diego, Cal.

These jetties fulfil nearly all the conditions laid down by Professor Haupt. Their principal point of variance is that they are not constructed on "compound and reverse curves," and have no gap or opening near the shore. In the localities mentioned these curves were not at all necessary, as there was no trouble in securing a deep channel inside the outer end of the jetty at either place. The bar, in advance of the jetty, was the real difficulty, and the channel across it was to be secured by directing the tidal currents upon it in a fixed location. In the instances mentioned this is accomplished just as well with the

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, p. 109.

Mr. Symons. straight jetties as it could be with curved jetties. In fact, it was necessary in almost every instance to protect the jetties against undue erosion by groins built out on the channel sides. The construction of the "breakwater," so called, at Aransas Pass, is not made clear in the paper. It is apparently a brush and stone structure, a large part of which is founded on sand in 6 to 8 ft. of water. Alongside this and close up, the channel has been eroded to 18 or 20 ft. in depth, according to the map. This condition might well excite alarm for the safety of the structure. The possible undermining may, however, be guarded against by some construction not alluded to in the paper.

Along our Northern Pacific Coast there is an average tide of 6 to 7 ft., and the storms are much more frequent and severe than in the Gulf, all of which, of course, makes the problem of bar improvement vastly different from that at Aransas Pass.

Neither was there, at any of these Northern Pacific Coast harbors, any need to leave a gap near the shore to permit the tide to enter freely. The jetties as located and built leave the door open almost to its full width for the entrance of the tide, and there is practically no choking down of the flood tide, which, theoretically, there is with parallel or converging jetties. Besides, at these Northern Pacific Coast points there is a very practical objection to leaving a gap near the shore. The rough and tempestuous character of the waters prohibits absolutely the building of the structures from floating plant, and they must be built from tramways extended out from shore. If the tramway should be built and a gap left in the brush and stone jetty built from it, it is certain that the unprotected portion of the tramway would be undermined and destroyed.

Judging from the map, Fig. 2, this cutting has commenced in the gaps left in the Aransas Pass structure, and it is quite within the bounds of possibility that this may extend, and channels develop therefrom, to a degree which will lessen seriously the flow in the desired channel. If the flood enters freely through the gaps, the ebb will also run out freely, and it would seem the part of wisdom, where there is so little tide as at Aransas Pass, to control as much of it as possible. The sill, *G E*, Fig 1, would at least be necessary, in the writer's judgment.

In the case of Grays Harbor, the natural bar channel had an apparently unswerving tendency to move from north to south, keeping on with this movment until its extreme southerly position was reached, then breaking out again to the north and taking up its slow movement to the south. The jetty was located to interpose itself against this southerly movement with the hope and belief that the pressure to the south would continue, and that this pressure on one side opposed by the jetty on the other, and the inflowing and outflowing tidal currents, would produce the necessary agitating and scouring current-effects

necessary to determine and maintain a good deep channel and direct it permanently over the bar in the selected location. This work was planned in 1894-95, and is truly a "reaction breakwater" in all essential features, as described and named by Professor Haupt. Whether it will fulfil all that the writer hopes for it remains to be seen, and can only be determined in the future. Mr. Symons.

The great Columbia River jetty, planned in 1882, is also such a "reaction breakwater," interposed against the tendency of the bar channel to move to the southward.

At Coos Bay the movement of the bar channel was to the northward, and, consequently, the jetty was interposed against it on the north, and this northerly tendency and the reaction of the jetty have determined and maintained an excellent, deep-bar channel, permanent in location, ever since the jetty was carried out sufficiently far to make its influence felt. The writer is unable to believe that, in the great majority of cases, the curved trace, indicated by Professor Haupt as essential, is at all necessary, or the best. There may be cases, however, where such a trace best lends itself to the situation and control of the forces at work. Each particular case must be judged by itself.

While the writer is a believer in the single-jetty system, wherever applicable, he also believes that there are cases where one jetty would not be sufficient, and where parallel or converging jetties are necessary. A safe rule to adopt, and one by which the writer was governed in designing all the later works of harbor bar improvements which he has had in charge on the Pacific Coast, is to locate the first jetty to be built as if it were the only one to be built, and hope for the best. Under this plan, if it be found that the results obtained are not satisfactory, then other works, a second jetty, perhaps, can be added to better control the natural forces and direct the flowing waters. The writer's experience causes him to be very chary of predicting absolutely the results which can be obtained from any structure built to control the waters and sands of an ocean bar.

Professor Haupt and all concerned are to be congratulated on the results obtained at Aransas Pass. The very marked improvement in such a short time mentioned, however, naturally causes one to ask whether there is a certainty that it is permanent. This is mentioned because, in conducting harbor works on the Pacific Coast, very extraordinary results were obtained at times, but they were not permanent, and the real value of the works could only be determined by finding out the minimum depth and width of channel which could be depended upon under all circumstances. The author recognizes this uncertainty by his statement that "modifications, however, are always to be expected in depths due to variations in the natural forces." These modifications may in time, and probably will, render the entire

Mr. Symons. or partial closing of the gap in the jetty, and the construction of groins for its protection advisable and necessary.

If the author had extended his studies to the works of harbor improvement designed and carried on by the Corps of Engineers, U. S. A., on the Pacific Coast, his claim as to the epoch-making character of the Aransas Pass structure would probably have been greatly modified, in view of the fact that structures practically identical in character were designed, located and built years before it was thought of.

The correct principles of the single-jetty system appear to have been applied very successfully at Aransas Pass, and Professor Haupt has invented a new name for such a structure. The writer cannot, however, believe that the engineering world will accept the name "breakwater" as properly applicable to the structure at Aransas Pass. It is not designed to and does not perform the functions of a breakwater as ordinarily understood and defined. It is a training wall or directrix to the flowing waters, and the name jetty fits it much better than the name breakwater, and is more in accord with good usage and common understanding. A breakwater is from its very nature a reactionary structure, its object being to deaden the force of the waves, which roll in from ocean or lake, by interposing its weight and bulk in their path, reacting against them. To speak of a "reaction breakwater" is a redundancy.

Mr. Ripley. H. C. Ripley, M. Am. Soc. C. E. (by letter).—The adoption of the reaction or curved breakwater, for the improvement of a harbor entrance, is without doubt the greatest advance made in the science of river and harbor engineering that has ever been recorded in this country. For this reason this paper will be read with especial interest by those engaged in this branch of the profession.

The principles herein applied, however, are as universal and well established as the law of gravitation itself.

These principles are:

First.—That of reaction, by means of which, water, constrained to flow in a curve, excavates and maintains a greater cross-sectional area of channel than when flowing in a straight course, and the ratio of depth to section is always greater in a curved than in a straight channel, under otherwise similar conditions. For examples of these effects, see *Transactions*, Am. Soc. C. E., Vol. xxiii, page 153, where it is shown that for the Brazos River the average sectional area is $13\frac{1}{2}\%$ greater in bends than in reaches, and the average maximum depth is 58% greater in bends than in reaches.

Second.—That of centrifugal force, by which the particles of flowing water tend to hug the concave side of a channel. This principle makes it possible, by means of a single curved structure, to concentrate the ebbing waters of a tidal harbor or the discharging water of a

river into a single channel of limited width, and convey it across the bar as effectively as can be done by means of two parallel jetties, where the aggregate length is double that of the single curved structure. Mr. Ripley.

Numerous examples of natural concentration of flowing water into channels of limited widths by this principle are to be found in the bays and sounds along the Atlantic and Gulf Coasts. A noted example is that of the channel of Galveston Harbor along the city front. This channel has a width of less than 1 000 ft. between the 6-ft. contours and a maximum depth of over 30 ft. This concentration of the tidal flow is due to the curved form of the shore line of Galveston Island, which brings into action the principle of centrifugal force.

These two are the cardinal principles involved in the application of the reaction or curved breakwater to the improvement of a harbor entrance. The writer thinks that no one will question their universality. Nor are these principles of recent discovery. They were recognized more than three-quarters of a century ago, by engineers having to do with harbor works.

The improvement of the entrance to the harbor of Swinemünde, Prussia, on the Baltic Sea, is an example of the successful application of these principles by means of a curved jetty extending from the shore on the east side of the entrance, across the bar, a distance of 3 900 ft., with a somewhat shorter angular jetty upon the west side of the entrance.

General Gillmore, in describing this work in 1881, wrote:*

"Several harbors on the Baltic Sea have been improved by means of jetties.

"Perhaps the most successful example of this method of improvement is shown at the entrance to the harbor of Swinemünde, the outport of the City of Stettin, an important commercial town of Prussia. * * *

"The west pier or jetty was designed to arrest the progress of the sand brought to the entrance by the littoral current, besides confining the currents of the outflowing waters. * * *

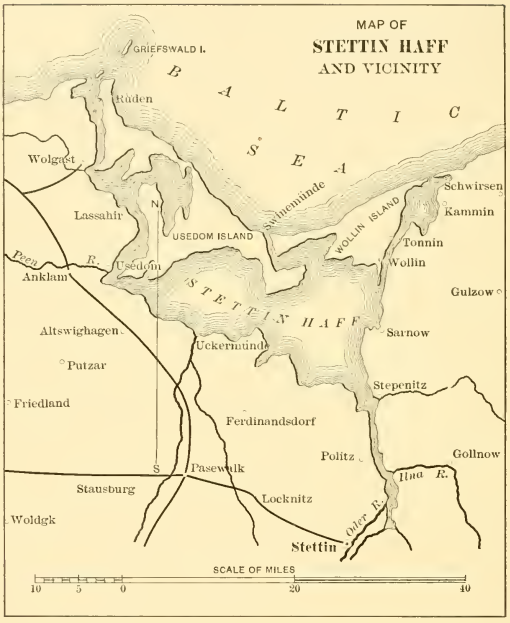
"The well-known fact that near concave banks, the currents of flowing water generally create and maintain a deep channel, induced the designer of the Swinemünde jetties to construct them on a curved plan, to secure ample depths, at least, on one side of the entrance channel. * * *

"The curved plan proved entirely successful, inasmuch as considerable depths were produced next to the east jetty; in fact, the opinion has been expressed that a more gentle curvature might have been better, as the depths in the concave bend are actually greater than needed, while on the opposite side, extensive shoals are found, necessarily reducing the width of the navigable channel and requiring the frequent use of dredging machines. * * *

"As a general result of this improvement, it is reported that the

* Report, Chief of Engineers, U. S. A., for 1881, p. 1060.

Mr. Ripley.



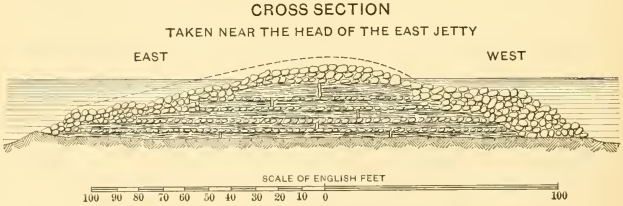


FIG. 3.

Mr. Ripley.

PLAN SHOWING

JETTIES OF THE HARBOR

OF SWINEMUENDE

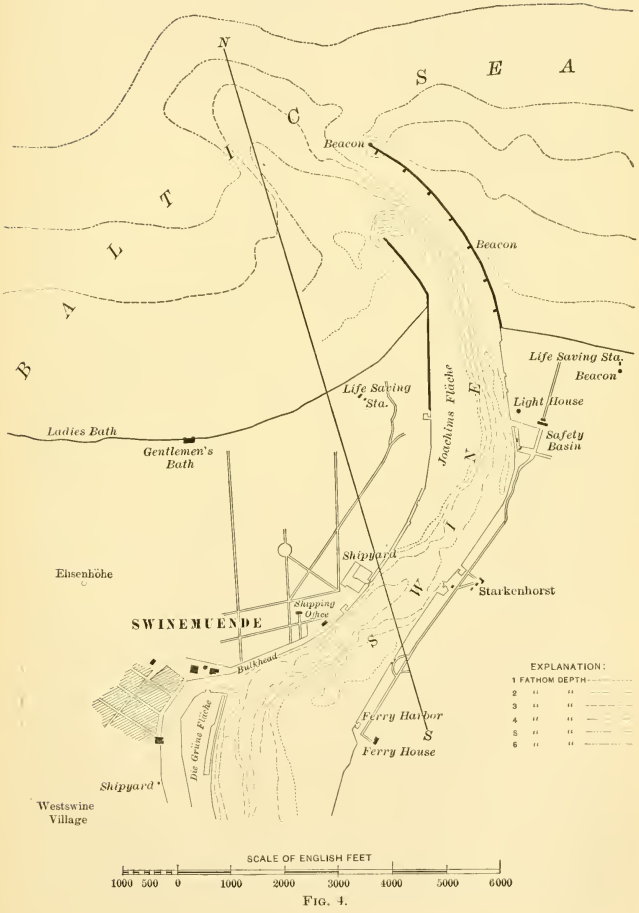


FIG. 4.

Mr. Ripley. principal channel, in which the depths were only 7 to 7½ ft. before the works were begun, now has a navigable depth of more than 20 ft., which has been maintained for nearly half a century."

The plan of this work is shown in Figs. 3 and 4. While it is stated that the "curved plan proved entirely successful," the work as a whole has not been so, from the fact that the curved jetty was placed upon the lee side of the littoral drift, which encouraged its accumulation on the channel side of the curved jetty, with a resulting growth of the bar seaward, which will necessitate the ultimate extension of the works to prevent its overlapping the outer end of the jetty. Had the curved jetty been placed upon the windward side, the littoral drift would have accumulated next to the shore and its seaward growth would have been much slower.

About 16 years ago, the writer designed a detached curved breakwater for the improvement of the entrance to Galveston Harbor. Its location is shown by the heavy broken line in Fig. 5. The line of the North and South jetties as constructed is also shown. The condition of the bar and channels is that of the time of the survey in 1884. The present position of the shoalest part of the bar is outside of the line joining the outer ends of the jetties. At this point, the resultant littoral movement is to the southwest, as is indicated by the gentle curve of Bolivar Channel to the south as it approaches the bar.

The breakwater is placed on the windward side of the channel, and its curvature and position conform to the curvature and alignment of the natural channel, and as a resultant of all the forces acting upon it. The discharging water from the tidal basin meets this curve tangentially, and, upon the principle of centrifugal force, will be constrained to follow the curve throughout its length. The channel will therefore have no greater width than at the point of meeting the curve, but, on account of its curved form, somewhat greater depths will be developed and maintained than would be the case if the same movement of water were confined between parallel straight walls.

Its position on the windward side of the channel is such as to intercept the littoral drift and protect the channel from its degrading effects; and the littoral movement, being normal to the breakwater at its outer end, will oppose no dynamic resistance to the ebb discharge. The existence of this breakwater would exert the least opposition to the admission of flood water to the tidal basin consistent with the control of the ebb discharge, and it is probable that, with the development of the channel across the bar, an actual increase in the tidal prism would result.

This design was approved by the United States Engineer Officer then in charge of the work, and its adoption recommended. The results at Aransas Pass have demonstrated fully that, had its construction been authorized, there would have been saved to the Government,

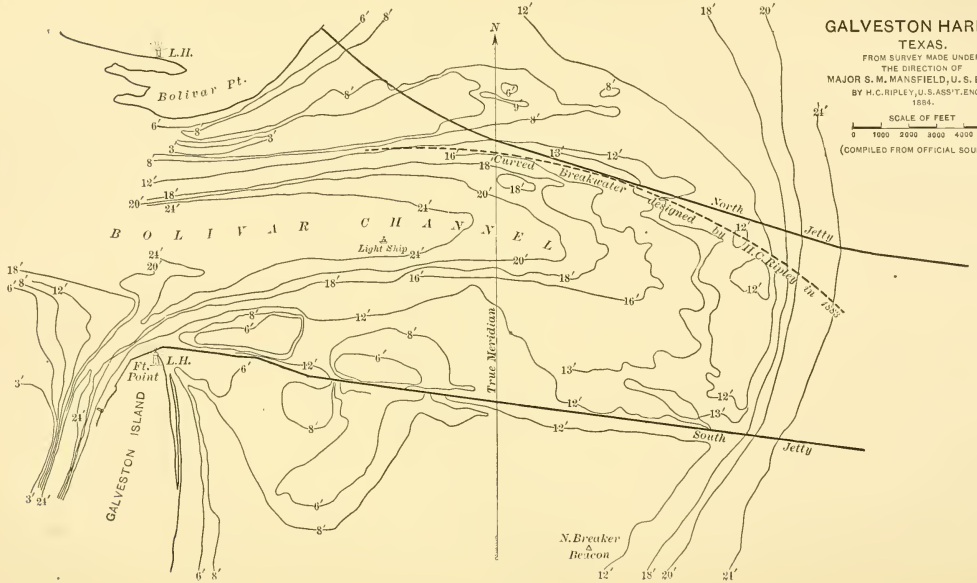

GALVESTON HARBOR,
TEXAS.
FROM SURVEY MADE UNDER
THE DIRECTION OF
MAJOR S. M. MANSFIELD, U.S. ENG'RS.
BY H.C. RIPLEY, U.S. ASS'T. ENG'R.
1884.
SCALE OF FEET
0 1000 2000 3000 4000 5000
(COMPILED FROM OFFICIAL SOURCES)
L.H.
Bolivar Pt.
BOLIVAR CHANNEL
Light Ship
Curved Breakwater designed by H.C. Ripley in 1883
North Jetty
South Jetty
True Meridian
Ft. Point
L.H.
GALVESTON ISLAND
N. Breaker Beacon
N

FIG. 5.

Mr. Ripley. in the improvement of this one harbor, not less than $4 000 000 in money and about four years in time.

There are many harbors on the Atlantic, Pacific and Gulf Coasts needing improvement, where the application of the reaction breakwater would be entirely feasible.

The curved jetty, involving the same cardinal principles as the reaction breakwater, would also be applicable at the mouths of many rivers, to the great saving in the cost of improvement.

After much study of the local conditions at the mouth of the Coatzacoalcos River, Mexico, the writer has proposed a single curved jetty for the improvement of the entrance to that river, with the estimated saving of about $1 000 000 in gold as compared with the estimated cost of two parallel jetties.

With the results at Aransas Pass, the writer is entirely confident that the improvement of the entrance to the Coatzacoalcos River would be entirely successful with the construction of the single curved jetty proposed.

Mr. Pitts. THOMAS D. PITTS, Jun. Am. Soc. C. E.—The speaker had the good fortune to be engaged as Assistant Engineer on the works at Aransas Pass for over two years, and, naturally, he has been much interested in the results.

From the depths maintained in the gorge, it would seem that the ebb-current velocity and tidal volume are sufficient to maintain a depth of 20 ft. or more on the bar, if properly controlled and directed. The speaker believes the results so far obtained show that the curved jetty on the north side of the channel, which was designed by the author, would be amply sufficient to control and direct the current, if completed.

In reports made to the Chief of Engineers, U. S. A., regarding contemplated improvements at Aransas Pass, it has been said that it would be very difficult to build and maintain a jetty there on account of the quicksand foundation. The curved jetty, built by the Aransas Pass Harbor Company, is founded on mattresses made of fascines of live oak and bay brush, the fascines being about 9 ins. in diameter and of varying length, according to the width of foundation desired. The mattress sinks into the sand somewhat when first laid and compresses considerably when fully ballasted, but it seems to last for an indefinite time. The foundation of the south jetty, built by the Government about 1885, was a similar mattress, and during Colonel Goodyear's blasting operations on the site of that jetty fragments of the mattress arose to the surface in considerable quantity. The wood of these fragments was quite sound, but the bark was partially decayed. A portion of the north jetty has now been in place for over 4 years, and no work of any kind has been done for more than 3 years. During that time there has been no evidence of any settlement of the

foundation, the small amount of subsidence noticed being amply ac- Mr. Pitts.
counted for by the consolidation of the stone in the body of the jetty, and by loss from the top due to heavy seas. In this last respect, the speaker has been surprised that the jetty has stood so well, in view of its exposed position, and the fact that no repairs whatever have been made.

Between August 1st and December 31st, 1895, a portion of the jetty 1 250 ft. long, *B*—*C* (Fig. 1), was completed to full size and section; a mattress was laid from *B* to *F*, 1 000 ft., and from *C* to *D*, 1 500 ft. The channel depths increased rapidly until, in November and December, a channel of 13 ft. could be traced across the bar. At about that time the foundation of the old Government jetty was uncovered, and the increase in depth ceased. Although the removal of a portion of the old foundation was urged repeatedly by the Consulting and Chief Engineers, no funds were provided for that purpose; but during the spring of 1896 the portion *C*—*D* of the jetty was raised to the level of high water, and the foundation was laid for a distance of 1 400 ft. from *D* toward *E*.

During August and September of the same year the mattress was extended to *E*, and the jetty was raised to low-tide level between *D* and *E*; but it was built of much less than full section, and two gaps of a total length of about 500 ft. were left. Until this last work was done between *D* and *E* the ebb discharge was practically not controlled at all, most of the flow passing to the northward of *D*, and the current across the bar was scarcely appreciable.

The condition of the jetty in the spring of 1896 was such that it caught and caused to be deposited on the bar the sand carried along the shore by the northerly current which prevails at that season. This current was of unusual strength and duration that year. In consequence of this sand deposit, the depth on the crest of the bar decreased so that there was only 5½ or 6 ft. of water in September. As soon, however, as the portion *D*—*E* of the jetty was partially completed as described, the current across the bar was noticeably increased, and the channel depths began to increase slowly. This increase was helped materially by the explosion of dynamite on the bar by Colonel Goodyear; chiefly by the destruction of a part of the foundation of the old Government jetty, but also partly by the softening of the whole bar, caused by the shocks of the explosions.

The increase in depth, which began in the fall of 1896, has been continuous since; being slower, or perhaps even ceasing entirely, during the spring and summer, and much more rapid during the winter, when severe "Northers" are of frequent occurrence. At such times the current is very rapid, surface velocities as high as 6½ and 7 ft. per second having been measured by means of float observations. These were probably not maximum velocities, as the engineering force had

Mr. Pitts. no facilities for making current observations during the times of greatest discharge.

The speaker's latest information regarding the depths on the bar is by letter from the keeper of the Life Saving Station at the Pass. From this he learns that there has been but little change in the channel during the past summer, but that the depths on the outer slope of the bar have increased somewhat. In the absence of a complete survey, it is not safe to assume too much; but the increase in depth on the outer slope would seem to show that the jetty is serving one of the purposes for which it was designed, *i. e.*, to prevent the advance of the bar seaward.

The south jetty, built by the Harbor Company in 1892, as mentioned by the author, was carried out some distance during the summer, and was apparently successful for a time. A deep channel was formed along the north side of the jetty, and there was some increase in the depth on the bar. The improvement was lost, however, as soon as the southerly littoral currents set in, in the fall. There was no improvement the next year, and a large part of the jetty had disappeared by the summer of 1895. This jetty was built of wooden caissons about 7 ft. in diameter, sunk in the sand and filled with sand and rock. A portion of this jetty is shown extending eastward from "Co. Wharf" on the map (Fig. 1). As will be seen, it did not in any way control the current, serving merely as a protection against the sand carried along the shore to the northward. In fact, the ebb current is deflected to the north by the revetment laid by the Government between "Co. Wharf" and Turtle Cove, and trends away from the shore line of Mustang Island before reaching the inshore end of the south or Nelson jetty.

The curved north jetty was designed by the author, not only to protect the channel from the inflow of sand from the north, which movement the speaker considers more important than that from the south, but also to control the current and compel it to scour out its own channel. The speaker does not believe that a straight jetty would serve the same purpose in this location, as suggested by Major Symons, for he thinks such a jetty would only deflect the current, without concentrating it, and that it would then spread out, fanwise, and lose its velocity and strength. With the curved jetty, on the contrary, the change of direction is gradual and continuous, and the conditions are similar to those in the bends of a rapid river, where it is well known that the greatest velocity and depth is found next the concave bank.

Major Symons also mentions four harbors on the Pacific Coast where single, straight jetties have been used with gratifying results. The similarity of the cases does not appear to the speaker, however. In one of these instances, the mouth of the Columbia River, there is

a very large fresh-water flow, and in all of them the tidal flow is many times as great as that at Aransas Pass. In all of these cases the entrance proper, between the headlands, is wide and deep, as shown by maps published in the Reports of the Chief of Engineers, U. S. A.; and there is a rise and fall of tide of 5 or 6 ft. The conditions are such that the bays doubtless fill completely at each tide, and the single jetty does not contract the entrance so as to diminish the tidal inflow. The depths and widths maintained in the gorges are far more than sufficient for any possible commercial needs; hence it is enough to control a part only of the ebb flow. Mr. Pitts.

At Aransas Pass, on the other hand, the entrance is naturally comparatively narrow, and the average rise of tide is only about 0.8 ft. It is therefore necessary, not only to control as large a part of the ebb flow as possible, but also to give the most ample opportunity for the flood tide to enter. The purpose of the opening left from E to G (Fig. 1), was to give a greater width of flood tide entrance, and the speaker believes that it is of material value for that purpose. It is true that a considerable quantity of water is lost through G—E during ebb discharge, but here again the water is thrown against the Mustang Island shore by centrifugal force, and the loss is limited to a surface one only. This is proven to the speaker's satisfaction by the fact that since September, 1896, when the foundation was completed to E, and the jetty from D to E was brought to a partially effective condition, there has been no tendency to scour in this opening; in fact, the depths have decreased somewhat.

The conditions at Tampico and at the mouth of the Brazos, where improvements have been made by means of parallel jetties, are radically different from those at Aransas Pass, on account of the large fresh-water flow at both of those places.

Mention has been made of the probability that shoaling would take place at the "cross-over." There would doubtless be some shoaling at that point, but the speaker does not think that the depth there would be the governing depth. He thinks the loss of water over the shoal to the southward would cause the governing depth to appear in the concavity of the jetty, and that it would not be less than 20 ft. under any circumstances, and probably much more. The latest chart available, from a survey made last winter by the United States Coast and Geodetic Survey, shows depths in the middle of the "cross-over" of about 17 ft. The speaker believes that the present bar acts as a dam to check the ebb current and to prevent scour on the inner slope, and that with the opening of a clear waterway of 20 ft. depth across the bar, the depths in the "cross-over" would be increased materially.

The fact that a current velocity which is abundantly sufficient to transport a given material will not always scour it from its bed was well shown at Aransas Pass. The material along this coast is a round-

Mr. Pitts. grained sand of light weight, which remains suspended in the water for a long time when once put in motion. Yet the speaker has seen this sand stand for days at a slope of 1 on 2 without perceptible erosion, notwithstanding the fact that a current of 3 to $3\frac{1}{2}$ ft. per second was impinging on it during every ebb tide. But the slightest shock, such as the explosion of a charge of dynamite under water in the vicinity, is enough to cause such a bank to slide. Any obstruction to the free flow of the water will cause eddies which produce rapid scour over a limited area. For this reason the speaker does not think that the stone from the foundation of the old Government jetty which was scattered by the dynamite explosions prevents scour. On the contrary, if that stone is as well scattered as he believes it to be, he thinks that it is a help rather than a hindrance, on account of the local scour produced.

The average range of the tides at Aransas Pass is less than 1 ft.; it is given on the Coast Survey charts as 0.7 or 0.8 ft. But there are times when the rise and fall are much greater. A long-continued southeast wind will raise the water 3 ft. or more above mean high water, and a succession of severe "Northers" will lower it as much below mean low tide. The speaker has seen the tide fall more than 3 ft. in less than six hours during a severe "Norther." As regards the character of the tidal curve; at the time of neap tides there is a well-marked diurnal rise and fall; but at spring tides there is but one pronounced tide in the 24 hours. At the time of spring tides, however, a short rise and fall frequently appears near the crest of the curve, and sometimes in the "valley." The foregoing is the general character of the curve, but it is by no means regular, the tides on this part of the coast being governed largely by the wind. The tide lags irregularly behind the moon's phase, and much more than on the open seacoast, but the speaker has no data at hand on this subject.

It is true that this work has stood for a comparatively short time only, as yet, but the speaker thinks that the results obtained since work on the jetty ceased, leaving it in an incomplete condition, are strong arguments in favor of the correctness of the theory on which it was designed.

Mr. Haupt. Lewis M. Haupt, M. Am. Soc. C. E. (by letter).—The writer feels under obligations to his colleagues for the interest and trouble they have taken in discussing this paper, each from his own standpoint and experience, it is true, but these side lights furnish an opportunity to illuminate the general problem.

General Review.—Some confusion has arisen from an attempt to measure results obtained elsewhere under totally different conditions; thus, instances are cited of the success of twin jetties where there is a large fluvial discharge at high velocities; cases which are not comparable with that under consideration, where there is no land drainage and a very small diurnal tide. Again, it is attepmted to compare a single

jetty projecting from the shore with the reaction breakwater which is purposely wholly detached therefrom; or, still further, linear work, having but a single function to perform, is cited as a precedent for curvilinear work designed to develop diverse agencies. Mr. Haupt.

Objections are also made to the use of the words "reaction breakwater" as being redundant, Major Symons not appearing to recognize the fact that the structure does break the waves from the north and northeast, forming a channel of quiet water in its lee and also producing a scour by the continuous reaction of the ebb currents along the concave front of the structure. These anticipated results, however, are so patent that no further space will be taken to reiterate them.

Mr. Corthell devotes considerable space to the impropriety of leaving an opening for the free admission of the flood tide, and quotes from Captain Eads the well-recognized principles applicable to half-tide or submerged jetties to sustain these views. As there is no question concerning their soundness and as the writer is on record in the papers referred to in his original paper, no time will be consumed in responding to this point which is also well covered by Mr. Marindin.

Mr. Corthell recognizes that the theory is "exceptional" and "peculiar," the work incomplete, and that, if correct, the cost will be only about one-third of that under present plans. Certainly, then, it is worthy of demonstration, which is all the writer has been seeking for years, and the testimony of Nature's agencies are now produced, as far as they have gone, to establish the correctness of their application at this difficult locality. This testimony does not seem to be refuted by any of the arguments submitted. The writer does not claim universal application of his plans to all conditions, as seems to be inferred from a few of the discussions; since each case is a special problem to be solved on its merits. Neither does he claim that a reaction is developed by a "convex bank." The function of the convex portion of the work is wholly distinct from the concave portion, but none the less important (as heretofore stated).

The unqualified statement that "it is not velocity * * *" that produces scour, is undoubtedly absurd, but it was not intended to be so used, as the context shows. It is a well-known physical fact that the mean velocity of a given stream, as generated by its surface slope, does produce remarkably different results under apparently similar conditions. The cases cited should suffice to illustrate the meaning, which was not to disparage either velocity or volume, without which no work could be done, but to direct attention to the tool, so to speak, whereby they may be applied to produce cutting. There is ample energy stored in the effluent at every bar, but, unless it is developed by some form of resisting medium and made to operate on the bottom, it glides to sea inoperative. The mean velocity is broken up and dis-

Mr. Haupt. tributed into complex bottom movements by the resisting breakwater.

A serious defect, so oft repeated in harbor works, in the writer's opinion, is the manner in which it is attempted to apply straight jetties to develop a channel; with the result that a deep hole is scoured out at the point of incidence of a current and the energy of the stream is dispersed by being reflected at an angle instead of being continuously supported across the bar. This is observable in many instances where twin jetties have been placed. It is the purpose of the curved breakwater to maintain a continuous reaction; which has been done, as shown by the results already secured. The currents are moulding their own channel, with a thalweg about 500 ft. from the axis of the work and parallel thereto.

Dublin harbor, which is regarded as one of the most successful instances of convergent jetty scour, was visited by the writer in 1889, and described in the paper on "Jetties for Harbor Improvements." Here the benefit was alleged to be due largely to the 600-ft. gap in the North wall, which admitted more water at flood than it emitted at ebb, thus changing the equilibrium between the heads of the jetties slightly in favor of the ebb. The dredging of the area within the jetties is not germane to the subject. From the relative positions of the jetties, shoaling is to be expected in this harbor.

Neither does the writer regard it as important to reopen the discussion on the resultant direction of the littoral drift at Aransas, which was brought out by Mr. Sweitzer's paper on the "Origin of the Gulf Stream,"* to which attention is directed. Suffice it to say that the accumulation of drift on every jetty built in the Gulf furnishes suggestive evidence as to the correctness of the location of the breakwater.

The writer does not wish to disparage any carefully observed data, but in these problems the elements are so varied in different parts of the same inlet and are changing so constantly (every hour almost) that even a very large number of simultaneous observations, extended over long periods, will not furnish as authentic a record as the chronographic resultant engraven on the plastic material moulded by the forces themselves. Careful comparative surveys are indispensable for this purpose, as well as measurements of the forces which have effected the results. A study of such charts, at various dates, is of more value than a few instrumental observations of velocities of currents, winds or tides, all of which, however, have a relative value. The writer's conclusions, as to the resultant direction of the littoral drift, were more fully set forth in previous publications. It is largely a resultant of the angle of wave approach, configuration of shore line, tides, wind, etc., all of which co-operate to produce the effect. At Aransas the consensus of opinion, as well as the efficiency of the work done, point to

* *Transactions*, Am. Soc. C. E., Vol. xl, 1898, p. 86.

the fact that the littoral drift is southwesterly. The small spur, 600 ft. long built in 1869, was a strong pointer. Mr. Haupt.

Twin Jetties at River Mouths.—The writer makes no issue as to the use of parallel jetties at the mouths of rivers, as his paper states. Twin jetties are but the logical outcome of Nature's laws in attempting to conduct an effluent stream over or through an obstructing medium. They constitute an aqueduct. To be most successful, they should confine the land discharge and exclude the tidal waters. The greater the preponderance of outflow, the better the result. But, in sedimentary streams, jetties serve merely to conduct the flow across the site of the original bar. When it reaches the sea, and the velocity is checked, the sediment is again dropped and the process of bar building and jetty extension must continue, unless there is a sufficiently strong littoral current to conduct the silt away from the outlet. If not, the depth may be maintained by dredging. Hence, as stated, every problem has its local features. A single curved jetty at a river mouth, however, may suffice to sustain the currents while crossing the bar, and prevent deposit, so long as they are reacted upon, causing their load to be thrust aside beyond the influence of the reaction and thus building up a mud or sand jetty at a suitable distance for navigation and automatically adjusted to the regimen of the stream, thus saving the price of one of the twin jetties.

This action is well illustrated in the results at Aransas, where the sand bar has been thrust aside and moulded without advancing seaward, making a counterpart to the breakwater, and aiding in confining the currents to their natural path. Other instances will be cited later.

Brazos Entrance.—As attention is called to the depths stated at the mouth of the Brazos as "erroneous" the writer begs to add that he hoped to find decidedly increased depths from the excessive fluvial discharge of last July's floods, and hence wrote to Texas to learn the results. A perfectly reliable authority replied, as stated in the paper, and on this condition a comparison was predicated. That it is substantially correct is confirmed by the letter of Mr. Dorchester, cited by Mr. Corthell, stating that it had shoaled to "15 ft." (It must have been at this date that the inquiry was reported.) "Subsequently deepened to 18 ft." The letter adds: "Bar entirely outside of jetties," etc. This, then, is still an incomplete work, and no final conclusions should be drawn. The writer has always had confidence in the twin jetties, as designed and partially built for this, the most promising river in Texas, for the application of this method. It is somewhat similar to Tampico, where Mr. Corthell's work has proven entirely successful. No "attack" is made on old methods, where properly applied, but it should be remembered that science is progressive, and in suggested improvements defects of existing methods may, with propriety, be pointed out, for the purpose of emphasizing

Mr. Haupt. and illustrating the particular applications of the new. This does not mean an unqualified condemnation of the old, and is in strict conformity with the law of evolution.

As early as November 19th, 1888, Mr. Corthell very kindly received the writer's publications and plans, returning them with the remark:

"They are all very interesting and you have certainly done an immense amount of work in collecting data relating to harbor phenomena. I cannot concur with you, however, in your improvement of tidal harbors, and cannot bring myself to believe that detached works such as you propose will obtain the depths required. * * * I have no occasion to design any works for exclusively tidal harbors."

This was before any results were obtained by any works of this character; but, for lack of space, further consideration of this valuable contribution must be deferred.

Mr. Marindin, physical hydrographer of the Coast Survey, recognizes the fact that streams, in carving out their channels in alluvium, ignore straight lines (which are those of the greatest slope between given points in a bend), and take to the curves where they scour to greater depths, as cited also by Mr. Ripley. This furnishes another answer to the point raised as to the velocity due to surface slope being the main factor in cutting channels, and brings out the reaction feature which has been so little utilized hitherto in engineering applications.

New York Bay.—In referring to the "Narrows" of New York Bay to illustrate the effect of "reaction *versus* velocity," as a cause for the observed effect, the writer quoted from memory impressions gleaned some 12 years before from the reading of a manuscript report by Professor Henry Mitchell, on the physics of this bay. This report was made in 1858–59, and revised in 1862, but was not published. The following extract therefrom is made by the courtesy of Professor Mitchell:

"The greatest depth of water was found at Station G (Fig. 6) in the Narrows, and here our lowest point of observation was 92 ft. from the surface. At this depth we found the flood current prevailing ten (10) hours out of the twelve (12) giving place to the ebb only at the most rapid fall of the tide. The flood stream, coursing along the bottom, reaches its maximum velocity near the time of high water; the highest rate observed being 1.82 miles per hour, a powerful scouring force! The greatest velocity of the ebb at the same point was observed to be 0.25 mile per hour, a force which according to the observations of Dubuat is incapable of overcoming the rolling friction of fine sand.

"The surface observations at this station indicate a very great preponderance of the *ebb current*, but it does not prevail so persistently as the flood in the lower stratum."

These resultants are more clearly illustrated by the diagram, Fig. 6, which accompanied this report, and which exhibit so forcibly the existence of this great vertical eddy swaying with the tide.

As the observations for velocity and duration were at a depth of 92 ft., while the bottom was 102 ft. below the surface, and the flood

resultant increases with the depth, it is probably nearer the truth to place its duration at 11 hours on the bottom, as stated in the paper. Mr. Haupt.

Is it safe to presume that a maximum velocity of 2.7 ft. per second, acting alone, will scour material from the bottom of a hole 102 ft. deep and will roll it up a slope to the level of the bed of the harbor, probably 60 or 70 ft. higher, when similar velocities will not disturb fine sand under 6 ft. of water on a down grade? Certainly, there must be some agency other than mere velocity to explain this phenomenon. The cautious engineer must take it into consideration, and may utilize it in designing his works.

Mr. Marindin is correct in his predictions as to possible extensions of the breakwater in case of extensive deposits beyond the end of the works, but as this is a drift and wave bar, with a practically clear

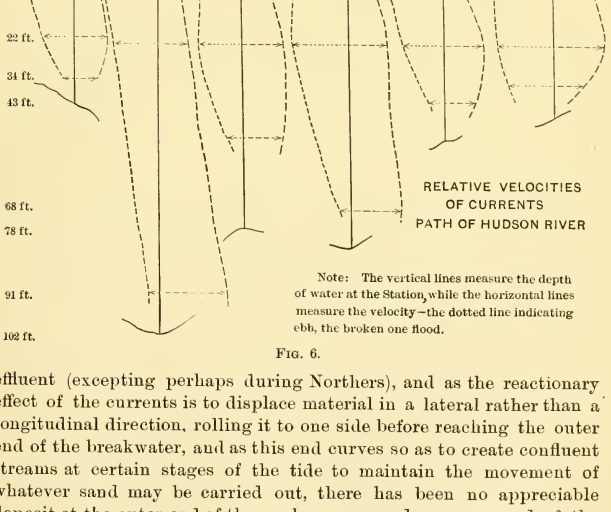


FIG. 6.

effluent (excepting perhaps during Northers), and as the reactionary effect of the currents is to displace material in a lateral rather than a longitudinal direction, rolling it to one side before reaching the outer end of the breakwater, and as this end curves so as to create confluent streams at certain stages of the tide to maintain the movement of whatever sand may be carried out, there has been no appreciable deposit at the outer end of the work, nor any advance seaward of the bar. The results speak for themselves. See discussion by Mr. Pitts.

In a sedimentary stream, a different curve and location should be adopted, to admit of greater possible extensions. That this is an important factor will be seen from the rapid movement of the Galveston bar, resulting from the failure to arrest the littoral drift by the construc-

Mr. Haupt. tion of the north jetty first; or still better, by the concave breakwater recommended by Mr. Ripley in 1883. The several efforts to secure deeper water at Aransas by a south jetty first also failed, because of the bar advance resulting in less depths. The effect is the same as that of placing a snow fence on the wrong side of a railroad cut.

The following is a statement showing the advances of the bar at Galveston, Texas, as indicated by the movement of the outer 18-ft. contour, and is taken from the blue prints of official surveys made on the dates indicated; measured from a "base" line joining station 220 + 18 on the North Jetty with the South Jetty beacon:

			From "base" to 18-ft. contour.
October	4th,	1894	650 ft.
"	23d,	"	750 "
December	"	"	1 150 "
March	3d,	1895	1 800 "
April	"	"	1 800 "
"	29th,	"	2 300 "
June	3d,	"	2 200 "
"	30th,	"	2 350 "
July	31st,	"	2 550 "
September	2d,	"	3 300 "
"	27th,	"	3 350 "

Depth in natural channel 16 ft. crossing south jetty. Over 60 000 ft. of jetty work in place in 1896. The increased depths in the channel were secured and maintained by dredging. The advance of the bar seaward during the year from October 4th, 1894, to September 27th, 1895, was, therefore, 2 700 ft. on a little more than half a mile, which was unprecedented. The bar, which was originally about 3 miles out, is now over 5, and still advancing.

The writer coincides entirely with Mr. Marindin's closing remarks as to the use of parallel jetties at the Brazos, while the case of the South or South West Pass is too large a problem to be embodied in this discussion, and must consequently be deferred.

Mr. Wisner distinguishes clearly the necessity of adapting the remedy to the conditions as to tidal or fluvial embouchement; also as to the need of admitting the full tidal prism at flood, and confining it during ebb to a local path, thus changing materially the conditions of equilibrium over that part of the bar intended for navigation, which cannot be done at tidal inlets by two tight jetties. The attempt to accomplish it has resulted in the waste of millions of dollars and constant charges for maintenance by sluicing or dredging.

While twin jetties have been successful at the mouths of rivers having no estuaries, the writer does not think, as already intimated,

that a single well-built and properly placed reaction jetty would not produce quite as good results where there is a resultant littoral current to remove the débris and prevent the advance of the bar. In fact, he is familiar with several very successful precedents. To cite only one, a single curved jetty of 3 000 meters radius has deepened its bar four-fold, where "for centuries before the struggle against the bar consisted in prolonging simultaneously two jetties from either bank of the river, leaving between them a distance of about 500 ft. But the sands outran the heads of the jetties, the flats were extended and the bar reformed. * * * Now (1889),* the bar has actually vanished," and the tonnage has increased enormously. From this instance it would appear that a single concave curved breakwater at the mouth of a river may produce the relief desired at much less cost than by two jetties, and hence becomes the better solution. This is also confirmed by the experience at the mouth of the Columbia River in Oregon, which was begun by Major Powell, Corps of Engineers, U. S. A., in 1885, at an estimated cost of $3 710 000 for a half-tide jetty, to be supplemented by raising it to high water, "or by building a short work on Peacock Spit for an increase of contraction."† The second jetty was not built, as it was found necessary to raise the first to high water to intercept the sand successfully, and this, with the drainage of the Columbia River, served to produce sufficient depth. Mr. Haupt.

Experience, thus far, at Aransas does not indicate that the short spur on the west side to the wreck of the *Mary* will ever be required, since the currents, even in the present condition of the work, are abundantly able to protect the channel from the eddy action referred to, and the fact that the breakwater is detached from shore modifies this action greatly. The currents have built their own protecting barrier of sand without cost to anyone or injury to navigation.

The jetties at Galveston, Cumberland Sound and other points are placed too far apart to co-act in producing scour. This results from the desire to admit as much of the flood tide as possible and therefore lessens greatly the concentration at ebb, and, since both flood and ebb movements follow the same path through the same area of cross-section, there is no beneficial resultant, and dredging becomes a necessity, while the bar moves seaward at a more rapid rate than before. This is not similar to the flood and ebb in a river where there is a resultant.

A few words as to Cumberland Sound, by way of illustration. The plan of 1879, as revised in 1891, consisted of two low jetties of stone, having their outer ends parallel and about 3 900 ft. apart. Work was commenced in 1881. The expenditure to July 1st, 1890, amounted to $479 550.28. The results were "considerable shoaling." The channel

* *Mémoire*, Société des Ingénieurs Civils, etc.

† In closing the discussion on Grays Harbor in 1896 it was stated that the writer was in error on the proposition to use two jetties at the mouth of the Columbia, and there was no opportunity to state the authority, which is, therefore, quoted as above.

Mr. Haupt. divided, the southerly one having "13 ft. at mean low water the same as reported last year."*

In reporting on this site, in 1879, General Gillmore said "the draught of water that can be carried over the Fernandina Bar (Cumberland Sound) varies from 11 to 12½ ft. at mean low tide, with exceptional depths 1 ft. greater, and a mean tide of about 6 ft." Estimated cost $2 071 023 to "give a low-water depth in the new channel of 20 to 21 ft."

The present depths between jetties, as shown by the chart of 1898† is less than 6 ft., and there has been appropriated $1 787 500. Hence it follows that for each of the, say, 6 ft. of depth lost between the jetties the cost has been about $298 000, while it was estimated originally that each foot of depth gained should have cost (estimating on 8 ft.) $260 000, making a difference of $558 000 per foot between the predictions and realities. The actual increase obtained by dredging south of the jetties, as reported, was 2 ft., making the cost per foot $893 750 for a temporary result secured after about nineteen years of effort.‡

On June 30th, 1897, the crossing on the bar half a mile south of the jetties gave "depths of only about 13 ft." Captain Carter then reported that the project has "proved unsuccessful for the sole reason that funds have not been supplied regularly and in adequate amounts," whereas, in the writer's opinion, the location and design of the jetties was largely responsible.

In submiting his revised plan, dated October 8th, 1890, Captain Carter recommended a concave wall on the north side of the channel, thus intending to dispense with the southerly jetty. This, he says, would involve "the removal of the central portion of the South Jetty, which is improperly located." For this plan, including the removal of a portion of the South Jetty, the estimate was $1 108 000, while for the original plan the estimate was nearly double; yet, after revision, "the Board, after consideration of all the conditions, does not advise any radical change in the general direction of the jetties as originally proposed. It does not think that a single jetty on the north side of the channel, curving gently to the south, would secure the deep water needed, but is of opinion that two jetties will be necessary," etc.

In view of this opinion it may be instructive to add that the last report available (surveys of June, 1898) shows only 5.9 ft. between the jetties, instead of 13 ft., while the present channel crosses the south one, the outer end of which serves to protect a crossing which has been opened on the bar by dredging under its lee; so that the two jetties are absolutely useless for concentration and scour, serving merely as sand arresters, and are serious obstructions to the currents which they cross.

* Report, Chief of Engineers, U. S. A., 1891, p. 1560, *et seq.*

† Report, Chief of Engineers, U. S. A., 1898, p. 1326.

‡ The cost of the dredging is not stated, but as it is not permanent and should be capitalized, say, at 3%, it will probably not differ greatly from $25 000, the interest on $833 333.

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The project was revised again in 1895, and estimated to cost $2 350 000; and in 1897 it was recommended that dredging be commenced in accordance with the adopted project, $50 000 being appropriated for the purpose.* Work on the jetties, by the Atlantic Contracting Company, was suspended on October 9th, 1897. The dredge boat could not work between the jetties because of the shallow depth, so it worked south of the South Jetty. A clam-shell dredge was also put to work to open a channel through the South Jetty to a depth of about 15 ft. The next season a channel of 17 ft. depth was opened by dredging the bar up to the jetty, but it shoaled up during the following months. The results secured, therefore, are wholly due to dredging, and are temporary in character.

It remains to connect the recommendations of Captain Carter, of a single curved wall or breakwater in October, 1890, and its rejection by the Board, with the plans of the writer, by stating that these plans were mailed to Captain Carter, then in charge of Cumberland Sound, at his request, on May 5th, 1888, followed by the discussion of the adverse report of the Board on the writer's plans,† March 28th, 1889, yet, after a careful study of the local conditions, in the light of the facts therein contained, he (Carter), without the knowledge or advice of the writer, concluded to recommend their adaptation to his problem, but which the Board declined to endorse, with the results as above. It is probably this action which Mr. Wisner characterizes as "the deadly wisdom of the Engineer Board * * * which proved fatal to the project."

The opening paragraph of Major Symons' contribution creates the impression that he is unable to distinguish, as was also the Board of Engineers of 1889, between a reaction breakwater, placed on the crest of the bar and having a specific form and position, and an ordinary jetty having its root on shore and projecting as a groin into the sea; but the distinction is radical and vital, and it would seem that the mere assertion of the lack of originality, as an opinion, should have but little weight when not supported by reference to the alleged precedents, which neither he nor the Board have seen fit to quote. Neither has the writer, after diligent search through an extensive field of harbor literature, been able to discover any anticipation, even approximate, save that of Major Ripley as proposed, but never built, at Galveston.

In view of the claims made in this part of the discussion as to the single-jetty work done on the Pacific Coast by Major Symons, the writer feels constrained to state that it is believed all of his work was planned subsequent to the publication of the plans and the discussions provoked thereby.

* Report, Chief of Engineers, U. S. A., 1898, page 1324.

† "Discussion of the Dynamic Action of the Ocean in Building Bars, being a reply to the Report of the Board of Engineers, U. S. A.," *Proceedings*, Am. Phil. Soc., March, 1899.

Mr. Haupt. The instance of Grays Harbor cited was not discussed until about nine years after the writer's earlier papers appeared, and if "it is in every essential respect such a reaction breakwater as described," as stated by Major Symons, then it does not furnish a precedent, but rather a remarkable confirmation of the soundness of the writer's plans, and becomes as well an infringement upon his property rights to his idea.

As the writer stated his views in the discussion of 1896, it will suffice to refer to the *Transactions** of this Society in reply as to the expectations; but the subsequent work done there will serve to show how little actual resemblance there is to the reaction breakwater, for the jetty is built on the south side, is connected with the shore, is straight for a considerable distance, whence it curves away from the channel (or is convex, not concave), is intended to be $3\frac{1}{2}$ miles long, is estimated to cost $1 000 000 and is expected to produce 24-ft. depths. In other words, it is not solely a tidal harbor; no provision is made for changing the equilibrium of ebb and flood or for the development and utilization of the reaction of the ebb currents, while only a partial barrier is opposed to littoral drift by this structure, which is so different in plan and location from that under consideration.

The ruling depth shown in the chart of 1896 was 13 ft., and that in the report of 1898 remains the same, while the proposed jetty crosses the channel. It would therefore seem to be a repetition of the unfortunate Government experience at Aransas Pass in 1885, excepting that the outer end of the jetty is convex to the channel instead of concave, which is so much the worse for the channel. At all events, since the jetty is not yet completed and no results are available, it does not seem to bear at all on the question of a precedent as to tidal scour resulting therefrom. This is confirmed by the admission that, after all, double jetties may be required there, and they are estimated to cost $2 500 000.†

The next reference given, as anticipating the writer's plans, is the single jetty at Coos Bay, located and built under the writer's direction and supervision, 1890 to 1895, which is a "reaction breakwater," etc. Here, again, there is a lapsus in the chronology, since the plans of the writer were published in 1887–88. How far, therefore, Coos Bay may be an anticipation or even an embodiment of those principles may be seen by a brief reference to its history.

The annual report of Captain Powell, 1886, says:

"The plan is to build a half-tide jetty or deflecting dike about 2 400 ft. long, * * * inside of the entrance, on a slightly curved line toward Coos Head. * * *"

The amount expended is $128 259.25. The original estimate of cost is $600 000.

* *Transactions*, Am. Soc. C. E., Vol. xxxvi, 1896, p. 109.

† Discussion by Mr. Allardt, *Transactions*, Am. Soc. C. E., Vol. xxxvi, 1896.

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This is, therefore, only a wing dam built on the bank of the stream inside of the bar. It was soon "submerged" and injured and "jetty operations were last closed in December, 1884."

At this point there is a rocky headland on the south and a sand spit on the north. It is said that "about once in 5 years the outflow sought a shorter line by a breach through the north spit," etc. Depth at low water, 14 ft. The report of 1898, made by Major Fisk, Corps of Engineers, U. S. A., says:*

"The present approved project† provides for constructing two, high-tide, rubble-stone jetties to obtain and maintain at the entrance to the bay a low-water bar depth of at least 20 ft. The North Jetty to be 9 600 ft. long and the South Jetty 4 200 ft. long."

It also provides for sand fences on the north spit. Estimated cost, $2 466 412.20, independent of the $213 750 expended on the original project. "All operations have been confined to the North Jetty. * * *" The channel was well south of the jetty, but it has since moved gradually back to its normal position nearly parallel to the line of the jetty.

The map accompanying the report of 1891 ‡ shows the plan to consist of two jetties, the north one extending from the sand spit southwardly, to canalize the effluent, then curving in a long sweep, convex to the channel and extending seaward in a straight, westerly line parallel to the companion jetty as proposed from Coos Head and 1 500 ft. distant therefrom. This chart bears the signature of Captain Symons, and is dated 1891. It contains the following legend:

"To construct two brush and stone jetties converging upon and crossing the crest of the bar at a distance apart of 1 500 ft. * * *"

Mean tide, 5 ft. 6 ins. Soundings in feet show a minimum depth of 19 ft., so that whatever results were secured at Coos Bay at that date were due solely to the renewal and extension of the Powell inner jetty of 1879, for which the twin jetties were substituted by the Board of 1889, and which were not in existence in 1891. Naturally, under this project, the proper order of procedure would be to build the North Jetty first, as was done, thus controlling the movements of the spit and protecting the channel; and this work, with the rocky headland, should suffice, if well placed, to maintain the requisite depth, with the assistance of the ample tide and rapid land drainage, so characteristic of the Pacific Coast. By this date, probably over a mile of this jetty has been completed, and to June 30th, 1896, the sum of $738 750 was appropriated, but the gain in depth resulting therefrom is not stated in this discussion. The Report of 1898 states, however, "During the past year the channel over the bar had a depth ranging

* Report, Chief of Engineers, U. S. A., for 1898, p. 2963.

† See Report, Board of Engineers, Officers, 1890, pp. 2936–65.

‡ Part v, facing page 3162.

Mr. Haupt. from 18 to 22 ft. at mean low water," while the Report of 1892 also states that at various times there were depths of 18 ft., and, when the channel was 1 000 ft. further south, there were 27 ft. at low water across the bar. As there were 19 ft. of water shown on the chart of 1891 before the North Jetty was started, it is difficult, therefore, to see wherein it has materially aided in deepening the channel by natural scour, even with the large volume of tidal and fluvial waters available.

In short, the North Jetty as built at Coos Bay is in no sense a reaction breakwater, such as is partially completed at Aransas Pass, and the writer fails to understand how a structure which did not exist in 1891 when the depths shown prior thereto varied from 19 to 27 ft. can be said to have developed a deep bar channel when the reported depths at date, 1898, were 18 to 22 ft.

Neither is the jetty at the mouth of the Columbia a "reaction breakwater," according to the writer's views and purposes. It is merely a sand barrier serving to arrest the littoral drift and thus reducing the work to be done by the ebb stream; but, as its outer end curves away from the channel, its reactionary effects are lost and its efficiency reduced. Moreover, the physical conditions as to its being exclusively tidal are wholly dissimilar.

The allusion to San Diego appears to be equally unfortunate as an attempt to establish a precedent. In 1891, it was reported that "The approved project contemplates the construction of a jetty about 7 500 ft. long on Zuninga Shoals, so as to give a depth of 26 ft. at mean low water over the bar, the present depth at the same stage being 21 feet."*

There was already another but crooked channel, having 36-ft. depths, and it was proposed to create a new one in a different place by building a breakwater from Coronado Island to cut off the drift and to deepen to "24 ft." by dredging.

Work on the jetty was commenced in September, 1893, and the latest report at hand, 1898, states:

"No work was done during the fiscal year ending June, 1898. There is now an available depth of 21 ft. at mean low tide. * * * Total cost to June 30th, 1898, $289 741.35."

Hence there is no improvement in depth and no similarity of conditions to those at Aransas, while the entire project was executed subsequent to 1887.

On this basis this analysis might be extended indefinitely, but the writer believes that his statement, as made, remains unshaken; that the incomplete work at Aransas marks "a distinct advance in the resources of maritime engineers"; and he appreciates the force of the concession made by Major Symons in his opening paragraph that "some of the most successful works of the Corps have been accomplished by adopting in the main the principles enunciated by Professor Haupt,"

* Report, Chief of Engineers, U. S. A., for 1891, p. 2961.

even though the references cited do not appear to show very great improvements, if any, nor an adaptation, to any considerable extent, of the principles. But the intentions are unquestionably good, even though the principles may not have been correctly applied. Mr. Haupt.

In reference to the alleged precedents on the North Pacific Coast it should be noted, moreover, that Major Symons states that the movement of the channel at Coos Bay was to the "Northward," and hence the jetty was placed on the north or far side thereof, while at Grays Harbor the "channel had an unswerving tendency to move from north to south, etc." "The jetty was located to interpose itself against this southerly movement with the hope and belief that the pressure to the south would continue." This jetty was, therefore, placed on the far or south side of the channel. Thus, in both instances, they are directly opposed to the location as adopted at Aransas, where the jetty was purposely placed between and not beyond the resultant littoral and the channel, with such decidedly beneficial results, while, in all other cases of Government work on the Gulf, the first jetty was constructed on the far side of the natural channels, with disastrous results and greatly increased cost.* This erroneous location, on the far side, increases the work of the ebb instead of diminishing it, by permitting the littoral forces to carry material into the channel instead of arresting it on the near side, which is a fundamental condition of the design and location, which does not appear to have been fully comprehended or applied by the engineers of those works. Yet the writer has laid it down as the first desideratum, viz., to keep the littoral drift from entering the channel. A jetty on the far side does not meet this condition, and is therefore no anticipation of either the principle or its application.

While claiming similarity, Major Symons goes on to show radical differences, and the reasons therefor. He calls attention to straight jetties built ostensibly to cause erosion and yet having "groins on the channel side to protect them from undue erosion," thus destroying in large part their efficiency. That the work at Aransas has withstood storms for over three years without material subsidence is a sufficient answer to the query as to how it was built. That was one of the purposes of its plan, and with the submerged Government jetty reflecting the currents against its face it may well excite surprise that it has not been undermined long ago. The gaps are undoubtedly a serious menace to the work. They were left by the Harbor Company and were beyond the control of the engineers, yet, notwithstanding the losses of water and entrance of sand through them, the deepening of the channel has been continuous. To build the sill *G E*, as suggested by the Board of Engineers, and endorsed by Major Symons, would, to

* *Vide* discussions on Sabine Pass, Galveston, etc., in *Journal*, Franklin Institute, and in *Proceedings*, Am. Soc. C. E.

Mr. Haupt. a large extent, destroy the preponderance of the ebb movements which the writer desired to create; cause the groin to fill with sand, the shore line to extend seaward, to deliver its drift in front of the channel, and re-create the bar in front of the breakwater. All of which are to be avoided as injurious.

The opening at the shore end was commended by H. L. Marindin, M. Am. Soc. C. E., Assistant on the Coast Survey, as early as December 26th, 1889, when he reported to the Superintendent, as follows:

"One of the good features in Professor Haupt's theory is his objection to the closure of beach or flood channels. This is a strong point in his system and it is in contradistinction to the plans generally submitted by engineers."

This strong point the Board's plan would destroy. Mr. Marindin evidently has seen no reason to modify this opinion since. In none of the cases cited by Major Symons is this salient feature present, neither is the concavity for reaction; nor the peculiar combination of curves intended to reinforce and protect the work from wave action on the one side or too great erosion on the other.

The writer does not claim that the combination, as partly built at Aransas, is a panacea for all ills nor that all its features are at all times "essential," but for tidal inlets he does believe that the results, under peculiarly adverse circumstances, stand unrivalled in the history of maritime works, and that its intelligent application will effect far more rapid and permanent results at much less cost than twin jetties as now applied.

If the writer comprehends clearly the drift of the argument submitted by Major Ripley it is to show that the principles embodied in the peculiar structure built at Aransas Pass are not novel, but, as the writer has made no claim to having discovered new laws or applied new principles, he need make no other comment than to call attention to the new embodiment of these old principles in the breakwater which was located here with the assistance of Messrs. Ripley and Wisner.

In the ancient harbor of Swinemünde referred to, however, there will be found some material differences between the curved jetties in pairs at the mouths of sounds having large fluvial water compartments with several inlets and the case in point. The extract from General Gillmore's report of 1881 shows two jetties, both curved and rooted to the coast line, which it is asserted were "entirely successful," yet "requiring the frequent use of dredging machines." This, certainly, is a very different construction and result from that at Aransas, where there has been a marked increase from a single detached structure, without dredging, and which has been constantly deepening.

Mr. Haupt.

The progressive improvement at Aransas, due to scour alone, may be seen from the following surveys and reports:

	Depth on Bar. M. L. W.
Survey of December 10th, 1896	6.5 ft.
" " February 2d, 1897	8.0 "
Old jetty breached (500 ft. in length) between December, 1896, and May, 1897:	
Survey of June 8th, 1897	8.75 "
" " November 5th, 1897	9.25 "
Reconnaissance, February 5th, 1898	10.00 "
Pilots reported, June 15th, 1898	11.00 "
" " August 29th, 1898	12.00 "
" " (by telegram) January 4th, 1899	13.3 "
United States Coast Survey, February 11th, 1899	15.0 "

These are the depths entirely across the bar, with depths on both sides of the old Government obstruction of over 22 ft.

The writer would be pleased to go more fully into the details of the harbor works on the Baltic, and make public some very interesting old charts, covering several centuries, secured during his visit to these ports, but the length of this discussion is already too great. Suffice it to submit a few excerpts from the report of Assistant Davidson, in 1875, showing the local conditions at Swinemünde. From this it appears that the longer mole is on the side to "windward" as regards the drift and not to leeward; there are also the usual phenomena attending jetties in pairs, but the channel is more permanent and more easily maintained because of its curvature being concave to the channel.

*Swinemünde on the Baltic.**—The Sound has three outlets. There is no tide in the Baltic save that created by winds. In 1828, the depth was 6 ft. There is a constant current out from Stettiner Sound, supplied by fresh water from the rivers Oder, Ihna and Ucker. The channel at 24 ft. depth is not quite 175 yds. (525 ft.) wide. Two ocean steamers can pass without difficulty.

"The artificial channel is composed of two nearly parallel moles of unequal length * * * curving well to the westward. The outer mole is 115° of a circle having only 2 000 yds. (6 000 ft.) radius, and for two miles the deep water scours directly under its wall."

The west pier has no parapet. The gales of 1874 and 1875 carried away 250 ft. of the east parapet bodily into the channel. The piers have been prolonged several times as the deposits have taken place. The beach has made 1 050 yds. seaward. What littoral drift there is seems to prevail from the eastward, and the deposit on the westward is partly from the Sound and partly from the littoral drift, but the increased depth is not wholly due to the piers, but in part to the con-

* Notes from the Report of George Davidson, Assistant, U. S. Coast Survey, 1875, p. 300.

Mr. Haupt. stant use of a small dredger. The cost is quite moderate and the results the most favorable yet seen upon a coast subject to heavy gales.

In conclusion, therefore, the writer is unable to find, in the cases cited, a solitary instance of a tidal entrance where there is a detached, concave, single, reaction breakwater, which has effected so great scour, in so short a time, with no assistance from dredging, in the face of an almost fatal obstruction, and at so little cost. If such a precedent is known to exist he would be grateful to learn of its location.

In view of the results already attained by this reaction breakwater and of the efforts which have been made, first, to prevent its introduction; and, subsequently, to so modify it as to destroy its practicability, he believes that the work should be carried to completion upon the original plan as a contribution to science and a demonstration of its ultimate efficiency. It is self evident that if equally good results can be obtained by so inexpensive a work, without dredging, many millions of dollars may be saved, while the barriers which have so long obstructed our commerce may be removed much more rapidly.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

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No. 864.

INSUFFICIENT PROVISION FOR COUNTER-STRESSES IN RAILROAD BRIDGES.

By HENRY S. PRICHARD, M. Am. Soc. C. E.

PRESENTED SEPTEMBER 20TH, 1899.

WITH DISCUSSION.

Many specifications for railroad bridges do not make sufficient provision for counterstresses. The term counterstresses must be understood, in what follows, to apply, not only to the stresses in the counters, but to all stresses from the action of the combined dead and live loads (including the effect of shock, sudden application, etc.), which are of a reverse character to the stresses from dead load alone.

In a good modern specification the unit stresses allowed are so low that, except for the counterstresses, a bridge designed and built in accordance therewith could reasonably be expected to be able to carry without serious injury a live load at least twice as great as the live load specified.

In view of the great increase which has been and is now taking place in the weights of locomotives and trains, not to mention other pertinent considerations, the wisdom of requiring this seeming surplus of strength will hardly be questioned, yet, as regards counter-

stresses, many otherwise good specifications make no such provision for increased loads.

This defect is probably due in most cases to the failure of the authors of the specifications to realize that a provision for possible increase in loading, which is adequate for all other stresses, may be entirely inadequate for counterstresses.

To assist in the consideration of this subject, the relations between loads and stresses are stated as follows:

Calling L the live-load stress, D the dead-load stress, A the sectional area, and S the stress per square inch, then for counterstresses $S = \frac{L - D}{A}$; for stresses other than counterstresses $S = \frac{L + D}{A}$.

From these equations the following propositions are evident:

For stresses other than counterstresses, if the live load is doubled, the stress per square inch cannot be more than doubled, so that if for the original live load the stress per square inch was less than half the safe stress, the stress per square inch would still be within safe limits if the load was doubled.

For counterstresses any increase in the live load must make a greater proportional increase in the stress per square inch. If the live-load stress is but slightly larger than the dead-load stress, then even a small increase in the live load may make a very large increase in the intensity of the counterstresses, while if the live load is slightly smaller than the dead load, a small increase in the live load may require counters where none were required under the original live load.

If the counterstresses in a bridge are obtained by simply subtracting the dead-load stresses from the reversed stresses for the specified live load, and the members subject thereto are proportioned by the specified stress per square inch, which is the way prescribed in many specifications, and a common practice among bridge designers, then such members will have a less factor of safety as regards increase in the live load than the other members of the bridge, and the trusses may in some panels have no proper provision whatever for any increase in the live load.

The absence of a regular provision for counterstresses does not necessarily result in immediate failure if the bridge is subjected to a load sufficient to produce counterstresses, as the stiffness of the chords and floor system, and the fact that the load is generally but a few

moments in the critical position which would produce the counterstresses may save a bridge from seemingly impending failure.

A case in point, brought to the author's attention about a year ago, is that of a 300-ft. span, shown in Fig. 1. This span, for an unknown period, had been carrying loads which, for want of counters, would temporarily buckle the main diagonals "A," whenever a loaded train crossed the bridge.

As this bridge is of a common type, and has no special features or unusual stiffness in the chords and floor system tending to supply the place of counters, it is fair to suppose that most bridges would possess some capacity for sustaining loads if some of the counters necessary to a proper design were omitted. It is to be feared, however, that loads so carried produce dangerous strains in the chords and floor system which may in time cause failure. Ability to carry loads when there is not proper provision for the counterstresses, like the ability to resist stress after the elastic limit has been exceeded, is a very desirable property in a bridge, and may, in an emergency, save it from disastrous failure, but it should not be relied on for sustaining the traffic, either present or prospective.

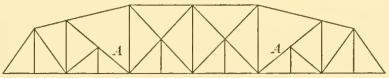

FIG. 1.

As a bridge designed in accordance with recognized good practice would be considered fairly safe as regards all stresses other than counterstresses, under increasing traffic up to a point where the live load was about double that originally specified, to be consistent, proper provision should be made for the counterstresses produced by a live load of at least double that originally specified. If the stresses are obtained from double the live load, the stress per square inch in members not subject to counterstresses will vary from something less than twice the specified stress per square inch in some members to nearly or quite twice the specified stress in others. Hence, it would seem to be consistent, after obtaining the counterstresses from double the specified live load, to allow nearly double the specified stress in proportioning.

All the previous references to live-load stresses are intended to be references to the live stresses, with the effect of shock, sudden applica-

tion, etc., included. The author's present practice in determining what members of a bridge are liable to counterstresses, and in proportioning them, is to add to the nominal live-load stresses (that is, the live-load stresses with the effect of shock, sudden application, etc., neglected), 125%, to combine the stresses thus increased with the dead-load stresses, and to use double the specified stress per square inch in determining the sectional area required.

Fortunately, even when the specifications require no provision for counterstresses beyond what would be produced by the specified live load, most bridge designers will make some provision, which, however, will differ greatly according to individual judgment. If the designers are contractors competing for work on a lump-sum basis, under specifications deficient as regards counterstresses, any provision they may make toward remedying such deficiency is a self-imposed handicap.

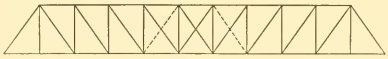

FIG. 2.

For bridges with parallel chords resting on two supports, as in Fig. 2, this handicap generally is not great, because for most cases only a few members will have counterstresses, even when the live load is doubled, and these stresses are mostly quite small; but for bridges with inclined chords, which for long and moderately long spans are now very frequently used, as in Figs. 1 and 3, the case is quite different, because the inclination of the chords decreases the dead-

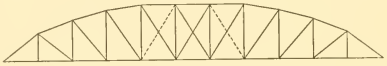

FIG. 3.

load stresses and increases the reverse live-load stresses in the web members, thus increasing the counterstresses. It is frequently customary to start the inclination of the chord at a sufficient distance from the center of the truss to avoid the use of counters in the portions of the truss having an inclined chord, and to make the inclination as great as the avoidance of counters and the maintenance of the required headroom will admit. As there is generally considerable

economy in making the inclinations of the chords as steep as possible, there is a decided advantage, from the commercial point of view of the aforesaid competing contractors, in making no provision for counterstresses in excess of the specifications.

The chief danger to be apprehended from specifications which do not make adequate provision for counterstresses lies, not in their use by those who from self-interest wilfully omit unspecified provisions which they believe to be essential to good design, but in their use by those who either fail to realize that the specifications are inadequate, or underestimate the extent of their inadequacy.

The same causes which lead engineers to write specifications which do not make sufficient provision for counterstresses may lead them to design bridges which are defective in this regard.

It is believed that what has been presented shows that provision should be made for counterstresses from some increase in the live load, perhaps more, perhaps less, than the increase advocated, and that whatever provision is decided on should in all cases be definitely and clearly stated in the specifications.

DISCUSSION.

Mr. Seaman. HENRY B. SEAMAN, M. Am. Soc. C. E.—The difficulty mentioned by the author arises from undue refinement in bridge design. The use of wheel concentrations implies that we are designing for some precise loading of the present time, while, as Mr. Prichard intimates, our designs should be an approximate provision for future conditions. The use of wheel concentrations practically submits the loading to the criticism of men who are not engineers, but who, nevertheless, exercise sufficient authority to restrict the loading. This condition would be avoided, to some extent, if a satisfactory equivalent load could be adopted.

The method suggested, of doubling the load, will avoid the difficulty described, though it seems to be rather an extreme measure. Fifteen years ago bridges were designed for 25 000-lb. concentrations, and to-day the actual concentrations in some instances reach nearly 50 000 lbs.; but this does not mean that they will reach 100 000 lbs. in the future. The larger part of the present increase took place between 1885 and 1890, and the increase since the latter date has been comparatively slight, though its adoption has been more general. The weights of locomotives having been increased about as much as the present gauge and car clearance will permit, any future increase will arise from a change of the distribution rather than from any material increase of concentration, and there seems to be no possibility of doubling the loads for which the heaviest bridges are now designed.

Mr. Jacoby. HENRY S. JACOBY, Assoc. Am. Soc. C. E. (by letter).—The subject of counter-bracing does not usually receive the attention it deserves in the design of bridge trusses. Sometimes the only rule followed is to put a counter in a panel on each side of those in which theory requires their insertion for the specified dead and live loads, without regard to the effect of impact or the future increase of loads. In some cases this may be sufficient, but in other cases the provision is wholly inadequate. This rule is certainly not a rational one.

The specification recommended in the paper is in accordance with the best practice, known to the writer, for the design of counters. The increase of 100% in the live-load stresses, besides the addition for impact, is not simply to provide for the gradual increase of the live load until it may reach that figure before the life of the bridge expires, but to provide for occasional loads of unusual magnitude, so as to avoid the necessity of shoring up bridges for such requirements, except those which may be nearing the end of their service.

As pointed out, the need for a better provision for counterstresses is greater in trusses with curved chords than for those with parallel horizontal chords. It seems that some account should also be taken

of the stresses due to the overturning moment of the wind, for a reversal of stress due to insufficient counter-bracing caused by the wind in conjunction with the live load may be just as serious in its results as if it were caused entirely by the live load. Mr. Jacoby.

Under the usual specifications, the stress due to the wind may be neglected in the design of any member unless its magnitude exceeds a given percentage of the dead plus live-load stresses, its character being the same. When, however, the character of the wind and live-load stresses is different from that of the dead-load stress it may require but a relatively small amount of the wind stress to cause a reversal of stress, and this fact often escapes notice on account of the too prevalent custom of failing to compute any minimum stresses.

Again, the addition of a counter-brace in a panel may, in turn, not only modify the magnitude of the minimum stress in an adjacent vertical, but may actually change the character of that stress, and thereby affect its design to a greater or less extent.

F. C. Kunz, M. Am. Soc. C. E. (by letter).—The fact that the counters are the weakest members of a bridge is justly claimed as one of the reasons for the preference of those specifications which, contrary to the fatigue formulas, assume a constant unit stress and add to the live load a certain percentage for the effect of impact. This percentage varies inversely with the loaded length, and is, therefore, greater for the counters than for the main diagonals, and greater for the latter than for the chords. The result is that the counters are comparatively heavier than the other members of the truss, and are sometimes necessitated in panels in which the fatigue formulas would not call for their use. It is evident that a portion of this percentage for impact can be considered as an increase of the live load itself, as long as we reduce the impact, for instance, by reducing the velocity of the live load, thus bringing the latter nearer to a static loading. The result would be, however, that the different members of the bridge would be subjected to different unit stresses. This is allowable for small variation of the live load, but, in the last few years, the increase in the weight of the rolling load has been extraordinary, as the following figures will show. In the beginning of this decade a great number of bridges were in service which were designed originally for a live load consisting of two engines of 55 tons each, followed by cars weighing 20 tons each, while the new specifications of that time called for a live load consisting of 100-ton engines and cars of 40 tons. At the present time, most railroads specify engines weighing 120 up to 175 tons (Lehigh Valley Mountain Engine) and cars weighing up to 50 tons and more, and there is no indication that the limit has been reached. Mr. Kunz.

In 1895, when the Pencoyd Iron Works were building a great number of bridges for the Missouri, Kansas and Texas Railway, they allowed

Mr. Kunz. in their designs—"Pencoyd Standards"—for a future increase of the live load in the following manner:

The unit stress of the Pencoyd Specifications for soft steel is 15 000 lbs. per square inch, but there is no reason why the material could not be stressed to two-thirds of the elastic limit, *i. e.*, 20 000 lbs. per square inch if necessary. It was determined what increase of live load, considering the impact, corresponded to an allowable unit stress of 20 000 lbs. per square inch in the chords. This increase depends, of course, on the ratio of the dead load to the sum of the live load and the percentage for impact, and is a minimum when the dead load = 0, in which case it is 33 per cent. The chords were selected because of their considerable weight, amounting to about one-half of the whole truss, and because they are, of all the truss members, affected the least by an increase of the live load, since the ratio of the approximately constant dead-load stress to the stress caused by the varying live load is the greatest. For ordinary plate-girders the allowable increase was found to be 35 to 38%; for a 100-ft. lattice girder, 40%; for a 150-ft. pin bridge, 41%; and for a 250-ft. pin bridge, 45 per cent. As an average, 42% was assumed, and, with a live load increased by this amount, with the corresponding percentage for impact and a unit stress of 20 000 lbs. per square inch, the web members and the floor were proportioned. This, of course, is exact only when the wheel spacing and the ratio of the wheel loads would remain the same.

The resulting increase in sections was comparatively small, and, as a rule, affected only the diagonals and counters, the floor and the connections, and in some cases necessitated the use of extra counters. For the details, unit stresses increased by one-third over their amount otherwise allowed have been used.

The Pennsylvania Railroad Specifications of 1897 contain a similar clause; it allows for 100% increase of live load and a unit stress of 30 000 lbs. per square inch, respectively, twice the unit stress otherwise allowed.

It is interesting to note that in Europe, although specifications using a percentage of the live load for impact which varies with the loaded length have been proposed (by Professor Melan, 1893), they have not been officially recognized as yet; some of the railroads using a more or less changed Launhardt-Weyrauch formula (France, Switzerland and part of Germany), others using a higher unit stress for longer spans than for shorter ones, but making no distinction between different members or between dead or live-load stresses (Prussia, Austria, Russia), and, again, others increasing the live-load 25 to 50%, irrespective of truss member or length of span (Prussia, Bavaria, Saxony). The most elaborate instructions concerning the proportioning of diagonals have been given in a circular of July 21st, 1899, by the Prussian Minister of Public Works:

Mr. Kunz.

First, where diagonals capable of tension only are used, provide counters in every panel in which a live-load increased by 50% reverses the stress of the main diagonal, the section as well as the number of connecting rivets of all the counters to be made the same as for the counters for the middle panel or panels. The counters are to be put in place after the bridge is swung, the holes for the connecting rivets on one end to be drilled in the field in such a manner as to give an initial tension.

Second, when stiff diagonals are used the counter stresses have to be determined with the original live load (without any increase), but, in proportioning the diagonals, the stresses of which can be reversed, use throughout the counter stress of the diagonals of the middle panel or panels, assuming only one diagonal as acting.

Mr. Baker.

Ira O. Baker, M. Am. Soc. C. E. (by letter).—In submitting the following, the writer has the feeling that possibly it is too elementary and too nearly self-evident to warrant its presentation in this place. However, since no treatise on bridges, which is known to the writer, even mentions the matter, and since two computers of two of the best of the smaller bridge manufacturers certainly do not know of the method, it will be briefly stated. The writer has used this method in his class-room for over twenty years.

In finding the stresses in the web members of a railroad bridge, due to a uniform load, it is apparently the custom to use the same load, whatever the proportion of the span covered, regardless of the fact that the equivalent uniform-load corresponding to the ordinary train increases as the length of load considered decreases. For example, assume that it is proposed to find the web stresses in a six-panel, 120-ft. span, due to a given engine and train. The equivalent uniform load for 120 ft., according to a standard diagram, is 4 545 lbs. per lineal foot of track; and this is the load to be used in finding the stress in the inclined end post. To produce a maximum in the tie in the panel next to the end, the load should cover four panels, or $4 \times 20 =$ 80 ft. The equivalent uniform-load for 80 ft. is 4 650 lbs. per foot, and this load should be used in finding the stress in the tie in the second panel. Similarly, the load for the third panel is 5 050 lbs. per foot, and for the fourth panel 5 670 lbs. per foot.

If the writer is correct in his belief that the above method is not generally used, then when the maximum web stresses are computed by using an equivalent uniform-load, they are generally less than the actual stresses, and the difference is greater in the counterstresses than in the main stresses.

Mr. Prichard.

Henry S. Prichard, M. Am. Soc. C. E. (by letter).—The writer does not share Mr. Seaman's belief that an intimate connection exists between insufficient provision for counterstresses and the use of wheel concentrations in designing bridges. At any rate there is nothing to

Mr. Prichard. prevent those who prefer the use of wheel concentration from having sound judgment regarding the proper provision for counterstresses. It may be that the present loads will not be greatly increased in the future, but the author cannot agree with Mr. Seaman that there seems to be no possibility of doubling the loads for which the heaviest bridges are now designed. At any time during the past 15 years there have been those who thought that the increase in loading had reached its limit. If the limit has now certainly been reached, then the provision proposed for counterstresses would probably be greater than necessary, but so also would be the provision generally made for all the other stresses.

As Professor Jacoby points out, an increase of 100% in the live-load stresses, besides the addition for impact, is not simply to provide for the gradual future increase of loads. It is in reality a factor of safety against any contingency in which the effect of the load on counterstresses is greater than would be anticipated from the normal loading. The author would not consider it wise to make this factor of safety as low as 42%, as Mr. Kunz stated was done in the cases he cites of bridges for the Missouri, Kansas and Texas Railway. If, in those bridges, the effect of the load was, from any cause, slightly more than 42% in excess of what would be anticipated from the normal load, then, as regards all stresses other than counterstresses, the effect would simply be to increase the stresses to slightly over 20 000 lbs. per square inch; but, for counterstresses, the effect might be to put counterstresses where no provision whatever has been made for them.

The writer prefers the provision cited by Mr. Kunz as given in the Pennsylvania Railroad Specifications of 1895. This provision, which consists of doubling both the live load and the allowed unit stress, was made at the writer's suggestion, after a preliminary draft of the specifications had been submitted to him for criticism.

Mr. Kunz's synopsis of European practice is interesting, but the instructions by the Prussian Minister, to which he refers, seem to be more complicated than scientific.

The influence of the wind in reversing stresses should be considered, as stated by Professor Jacoby, but, as the maximum wind pressure, in combination with the other conditions which tend to reverse stresses, does not occur frequently, it would seem to be justifiable, in providing for its influence, to use high unit stresses.

Professor Baker has shown conclusively that a bridge designed for a uniform load will have less provision for web stresses in general and counterstresses in particular, in comparison with the provision for chord stresses, than it would have were it designed for the usual engine and train load. While having no connection with the subject under discussion, it may be remarked, incidentally, that Professor Baker's analysis also indicates the difficult nature of the problem

which confronts those who are striving to find some simple substitute for the method of determining stresses from the engine wheel concentrations. Any one who is expert in the use of the moment diagram will find that for ordinary cases it takes less time and trouble to use the method of wheel concentrations than it would to use an equivalent uniform load if a different equivalent had to be determined for each different position of the load. Mr. Prichard.

MEMOIRS OF DECEASED MEMBERS.

CHARLES EDWARD EMERY, M. Am. Soc. C. E.*

DIED JUNE 1ST, 1898.

Charles Edward Emery was born at Aurora, N. Y., March 29th, 1838, and was educated at the Canandaigua Academy. His first work was in manufactories and in the drawing offices of railroad shops in the vicinity of his home, after which he turned, for two years, to the study of law, with the intention of becoming a patent attorney. Upon the outbreak of the Civil War, Mr. Emery organized a company of volunteers, but on the announcement that the company was not needed he sought admission to the Navy. He was appointed Third Assistant Engineer in the Navy, July 11th, 1861, and was promoted to Second Assistant Engineer, December 18th, 1862. Mr. Emery's service included blockading duty with the Gulf Squadron and various engagements with the Confederate fleet below New Orleans, battles with forts at Pensacola, Fla.; with Forts Jackson and St. Philip, the capture of New Orleans and fights with the forts at Vicksburg and Port Hudson. Following his promotion, Mr. Emery was transferred to the *Nipsic*, the first of a new class of fast gunboats, stationed in Dahlgren's fleet, off Charleston, S. C. Some suggestions made by him, as to experimental steam apparatus, led to his selection, by the Chief Engineer of the Navy, in 1864, for scientific research in the United States Navy Steam Expansion Experiments at New York. Mr. Emery's labors for the Government did not, however, cease with his resignation from the Navy in 1868, for later he served as Consulting Engineer and Chairman of the Board of Examiners of the United States Revenue Marine and the United States Coast Survey, and, dating from April. 1872, as a member of the special commission advisory to the Secretary of the Treasury in reference to the Revenue Marine service.

Following his retirement from the Navy, Mr. Emery was engaged, in 1869, for a year or more, in making, for the Novelty Iron Works in New York City, a series of experiments upon stationary engines which it was proposed to manufacture. The records of these researches were published subsequently by Professor W. P. Trowbridge, and entitled "Condensing and Non-Condensing Engines." It had been arranged that Mr. Emery should become general superintendent of the works, but, owing to a change of plans, the establishment was

* Memoir prepared by the Secretary from papers on file, and from various sources.

closed, and Mr. Emery entered into business on his own account, as a consulting engineer.

Mr. Emery served as General Superintendent of the Fair of the American Institute in New York City in the autumn of 1869, and directed the scientific work of that institution in the testing of engines and boilers. In 1876 he was appointed a member of the international jury at the Centennial Exposition, on engines, pumps and mechanical appliances, and was associate to committees on other scientific apparatus. He inaugurated an extensive series of tests of engines and boilers, and prepared extended reports of this work, which were subsequently published. In 1893, Mr. Emery was appointed one of the judges of the World's Columbian Exposition, on dynamos and motors. He served in 1884 on the committee appointed by the American Society of Mechanical Engineers, to prepare a code for steam boiler trials, and, in 1895, was made chairman of a committee to revise the code previously reported. He was engaged in the final revision of this committee's report at the time of his death.

Mr. Emery's advisory services for the National Government began in 1869, with his appointment as Consulting Engineer to the Coast Survey and to the Revenue Marine. His engagement with these branches terminated when they passed, successively, into the control of the Navy Department, in 1879 and 1891. He fitted out nearly twenty of the revenue cutters, being in some cases responsible for the construction of the hulls, as well as the machinery. His experimental work in this service, in the construction of economical engines and in the scientific determination of engine-efficiency, attracted much attention. In 1874, he was able to place three different types of machinery in three hulls of the same size: first, a long-stroke, high-pressure, condensing engine; second, a short-stroke, low-pressure, condensing engine; and third, a fore-and-aft compound condensing engine. These vessels, as well as a subsequent one, in which the cylinder of a high-pressure condensing engine was jacketed, were tested by a joint board of engineers from the Navy and Treasury Departments, and the published results of this trial, together with an analysis of them by Mr. Emery, were widely circulated and used. Soon after this, New York University conferred upon him the degree of Doctor of Philosophy.

Possibly the most notable of Mr. Emery's engagements in private practice was the work of the New York Steam Company, which he took up in 1879. This involved the generation of steam in a central plant and its distribution for heating and power purposes. In its construction, the plant was the largest of the kind attempted, and included pioneer work in many details. The design and superintendence of the construction of this plant were in Mr. Emery's charge until 1887, and, with its working, were described in a paper read before the Institution of Civil Engineers, in 1889. For that contribu-

tion, the Institution awarded him a Watt medal and a Telford premium.

Mr. Emery's practice as a consulting engineer and professional expert was extensive, and his services were in request in important legal cases. As consulting engineer for the City of Fall River, Mass., he was instrumental in bringing about a novel compromise of the difficulties between the city and the mill owners, resulting in an agreement whereby water was to be thereafter furnished to the city from the Watuppa ponds in consideration of the abatement of taxes on water power. He was a member of the commission on the purchase of the Long Island Water Supply Company's plant by the City of Brooklyn, N. Y.; was engaged in the matters of the water supplies of the cities of Newark, N. J., Lowell and Worcester, Mass.; the Holyoke, Mass., water-power tax cases, and had been consulting engineer for the Pneumatic Dynamite Gun Company. In 1887, he served as a member of the Board of Experts appointed to give an opinion as to the best method of increasing the number and size of trains on the New York and Brooklyn Bridge, by changes in the New York terminal, and at the time of his death he represented the Central Railroad of New Jersey in the matter of the Bound Brook floods.

Mr. Emery was elected a Member of the American Society of Civil Engineers May 6th, 1874. He contributed extensively to scientific literature and to the publications of the engineering societies in which he held membership. The papers by Mr. Emery published in the *Transactions* of this Society include: "Compound and Non-Compound Steam Engines, Steam Jackets, Etc.," December, 1874; "Connected-Arc Marine Boilers; Principles of their Construction," December, 1876; "Relative Quantities of Material in Bridges, of Different Kinds, of Various Heights," read at the Ninth Annual Convention, April 25th, 1877; "Steam Engine Economy," March, 1878; "Flexure and Transverse Resistance of Beams," June, 1879; "The Cost of Steam Power," June, 1883; "The Station B Chimney of the New York Steam Company," April, 1885; "A Novel Application of the Polar Planimeter," July, 1886; "Clamp for Pulling Sheet Piling," March, 1889; "Observations on the Forth Bridge," June, 1890, and "District Steam Systems," March, 1891. Mr. Emery was also a Member of the Institution of Civil Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the American Institute of Mining Engineers, the American Association for the Advancement of Science and a Fellow of the Brooklyn Institute of Arts and Sciences.

HENRY FLAD, M. Am. Soc. C. E.*

DIED JUNE 20TH, 1898.

Henry Flad was one of many men of rare ability for whose presence in America we are indebted to the Republican Revolution of 1848–49 in Germany. He was born July 30th, 1824, in the Grand Duchy of Baden, near the university town of Heidelberg. His father, Jacob Flad, dying within the same year, his mother, Francisca Brunn Flad, very soon afterward removed to the town of Speyer, a few miles distant, upon the left bank of the Rhine, in the Rhine Palatinate, a province belonging to Bavaria. After passing through the preparatory schools of Speyer, young Henry entered the University of Munich, in Bavaria, where he took the polytechnic course. Upon his graduation in 1846, at twenty-two years of age, he was given a position in the engineering service of the Bavarian Government, his first employment being on works for the improvement of the River Rhine.

The years which immediately followed, particularly 1848 and 1849, were years of great political commotion throughout Europe. Encouraged by the success of the revolution in France, which drove out King Louis Philippe, the longings of the German people for a freer and more united government found such vigorous expression that the princes of the many petty States into which Germany was divided acceded to the convocation of a National Assembly or Parliament, which, in May, 1848, met in Frankfurt to frame a constitution for United Germany. Unfortunately, the deliberations of this assembly showed such wide differences of opinion and so little ability to unite in any workable plan that the ardor of the more conservative classes began to cool. The princes seized their opportunity to reassert themselves, and repudiated the authority of the Parliament.

In Southern Germany, the champions of the Parliament took up arms in its behalf. Among them was Henry Flad, who, then in his twenty-fifth year, joined the Parliamentary army as a captain of engineers. Fortune, however, was against them, and, after several engagements, the Parliamentary army was driven into Switzerland, and disbanded. Meanwhile, its leaders were placed under the ban, and Captain Flad, with many others, was sentenced to death.

Under these circumstances he very naturally turned his face westward, took passage for the United States, and, in the autumn of 1849, landed in New York.

His first employment, after his landing, was as a draftsman in an architect's office. It was not long, however, before he entered the

* Memoir prepared by Robert Moore, Joseph P. Davis and J. A. Ockerson, Members. Am. Soc. C. E.

engineering service of the New York and Erie Railroad, then under construction, his headquarters being at Dunkirk, at the extreme western end of the road. Mr. James P. Kirkwood and Mr. James H. Morley, with whom Captain Flad was afterward associated, were also employed at this time on the same road. After the completion of the New York and Erie Railroad, in 1851, we first hear of Captain Flad as located for a time at Tonawanda, between Niagara Falls and Buffalo; and, then, in 1852, as an assistant engineer in the construction of the Ohio and Mississippi Railroad from Cincinnati to St. Louis, his headquarters being at Vincennes, Ind.

Upon the opening of the Ohio and Mississippi Railroad to St. Louis, in 1854—that being the first railway to reach St. Louis from the East—Captain Flad went to Missouri as an assistant engineer on the Iron Mountain Railroad, of which his former colleague on the Erie Railroad, Mr. James H. Morley, was the chief engineer. During the construction of this road, Captain Flad was located at Potosi, Mo. After its completion to Pilot Knob, where for a number of years it ended, he became land and tie agent of the railroad company, with headquarters at Arcadia, Mo. During this period, on September 12th, 1856, Captain Flad was married to Miss Reichard, of St. Louis.

Upon the outbreak of the Civil War, in 1861, Captain Flad came to St. Louis; and enlisted, on June 15th, as a private soldier in Company F, of the Third Regiment, United States Reserve Corps. He was soon promoted Corporal, and then Sergeant.

In July, 1861, a regiment, known as "The Engineer Regiment of the West," recruited mainly in the States of Illinois and Missouri, was organized by Colonel J. W. Bissell, and Henry Flad was made Captain of Company B. In August of the same year he was detailed by General Fremont, then in command at St. Louis, for service in the construction of fortifications at Cape Girardeau, Mo., where he remained for several months. Later in the year, when Fremont was succeeded by General Halleck, Captain Flad was ordered to join General Pope in southeast Missouri, and served as a staff officer through the campaign of New Madrid and Point Pleasant, and at the taking of Island Number Ten, after which he rejoined his regiment at New Madrid. He was with his regiment at Fort Pillow and Pittsburg Landing, and during the operations before Corinth. During the summer of 1862 he was engaged in repairing the Mobile and Ohio Railroad, in building forts at Corinth, and in repairing the Mississippi Central Railroad. He was also engaged in Grant's advance on Grenada. In February, 1863, he was ordered to Young's Point, where he was employed in engineering work, as he was later at Baxter Bayou, Lake Providence and Bayou Macon.

In April, 1863, he had charge of the repairs of the Memphis and Charleston Railroad at Memphis, Grand Junction, Jackson and

Columbus. In October of the same year he was employed in repairing the same railroad east of Corinth, under General Sherman, and was with him at Cherokee, Bear Creek and Iuka, in northern Mississippi.

Meanwhile he had been promoted on November 17th, 1862, to the rank of Major, on July 30th, 1863, to that of Lieutenant-Colonel, and on October 16th, 1863, to that of Colonel. On January 1st, 1864, at Nashville, Tenn., he was transferred, as Colonel, to the First Regiment of Engineers, Missouri Volunteers, a new regiment formed by the consolidation of the former engineer regiment and the Twenty-fifth Missouri Infantry. During the summer of 1864 he was engaged in completing the Nashville and Northwestern Railroad from Nashville to Johnsonville, and in constructing defensive works. In August he was ordered to Atlanta, and served there and in that neighborhood until about the first of November, his last work being the construction of a new line of fortifications at Atlanta.

At that time the term of enlistment of seven companies expired. The command of the remaining five companies then, under the army regulations, devolved upon the Lieutenant-Colonel, and the Colonel was mustered out November 12th, 1864, at Nashville, Tenn. His term of service had been three years and six months, during which time, with not more than a week's leave of absence, he had been constantly in the field.

Upon being mustered out, Colonel Flad returned to St. Louis, and began to look around for employment in his profession. In a short time the agitation for an improved water supply for St. Louis resulted in a State law authorizing the appointment of a Board of Water Commissioners, charged with the duty of making surveys and plans and constructing a new system of water-works for the city. Soon after the organization of the new board, in the spring of 1865, Mr. James P. Kirkwood, who had formerly been chief engineer of the Pacific (now the Missouri Pacific) Railroad, and had just completed the building of new water-works for Brooklyn, N. Y., was appointed chief engineer, and Henry Flad, chief assistant engineer.

Surveys and investigations were at once begun, and, by the end of the year, a plan was presented for new works with intake, settling basins and filter beds at the Chain of Rocks, and a distributing reservoir on what was then known as Rinkel's Hill, on Easton Avenue, near the present city limits. This plan received the approval of the Board of Water Commissioners; and, as subsequent experience has abundantly proven, was undoubtedly the best. But, besides running counter to some private interests, it involved such a large outlay and such a radical departure from the old plan that, on the part of many leading citizens, as well as the city authorities, it encountered overwhelming disapproval. The opposition finally became so great that the Water Commissioners were called upon by the City Council to resign. To

this demand they presently acceded, and, in July, 1866, a new board, committed to a new plan, was appointed. Meanwhile Mr. Kirkwood had been commissioned to go to Europe to study the subject of filtration, and Colonel Flad was left as acting chief engineer. In December, 1866, a revised plan, with intake and settling basins at Bissel's Point and a distributing reservoir on Compton Hill, substantially as afterward built, was presented.

Early in the following year, the act organizing the Board of Water Commissioners was amended, the number of members being reduced from four to three, and in March, 1867, a new board was appointed with Colonel Flad as one of its members. This position he held continuously by reappointment for eight years, or until April, 1875. During that time the new water-works were completed, and during the year 1872 were put into service.

While he was still acting as assistant engineer to Mr. Kirkwood, Colonel Flad made the acquaintance of Captain James B. Eads, who was at that time engaged upon plans for gun carriages and turrets. The rooms occupied by the Water Board being larger than they then needed, Captain Eads, upon his request, had been granted space in which to place a draftsman at work. This was followed by frequent discussions between the two men upon engineering questions, which led to a mutual recognition of each other's abilities and laid the foundation of a life-long friendship. When, therefore, in 1868, Captain Eads, as chief promoter as well as chief engineer, was ready to begin the work of constructing the great bridge over the Mississippi River at St. Louis, he very naturally tendered the position of chief assistant engineer to Colonel Flad. As the duties of the latter as member of the Board of Water Commissioners did not require all his time, this opportunity to take part in this most interesting and important work was gladly accepted, and he retained his connection with it until its completion in 1874. Some of the boldest features of this great enterprise, such as the method of erection without false work, were due to Colonel Flad.

During 1875 and 1876 he was engaged as consulting engineer in various works in conjunction with Mr. Charles Pfeiffer, who had been associated with him on the St. Louis bridge, Mr. Thomas J. Whitman, Chief Engineer of the Water-Works, and Professor Charles A. Smith, of Washington University. Among his other engagements was that of engineer for the commissioners who purchased and laid out Forest Park, St. Louis.

In the autumn of 1876, the new charter of the City of St. Louis was inaugurated, and Colonel Flad was elected the first President of the newly constituted Board of Public Improvements. This office he held continuously for nearly fourteen years, being re-elected in 1881, 1885 and 1889.

The problem to which the new Board addressed itself was that of taking the whole system of municipal public works out of the mire of politics, and placing it upon the basis of merit and fitness. Into this work Colonel Flad entered with characteristic zeal and a determination which nothing could shake. His efforts were crowned with entire success, so that during the whole period of his administration the Board over which he presided had the entire confidence of the whole community. Every citizen felt sure that in every department of the public works the city received a dollar's worth for every dollar spent, and in this respect St. Louis became a model for other cities.

In April, 1890, he resigned his office as President of the Board of Public Improvements to accept an appointment as member of the Mississippi River Commission, a position in which he remained until his death.

In November, 1891, he was appointed a member of a committee of the Commission to prepare a project for the construction of a dredge boat to be used in deepening the Mississippi River over the bars in extreme low water—a problem in which his remarkable mechanical ingenuity found a fertile field. After a most careful study of the subject in all its bearings, plans were drawn for an experimental dredge, which was built under his personal supervision.

While this experimental dredge was by no means perfect, its main features were found to meet the requirements so well that they have been embodied in all the later dredges built by the Commission, and its construction marks the beginning of a new era in the history of river improvement.

Colonel Flad became a member of the American Society of Civil Engineers on February 15th, 1871, and was President of the Society for the year ending January 19th, 1887. He was also a charter member of the Engineers' Club of St. Louis, and for twelve years, from 1868 to 1880, was its President, thus receiving from both organizations the highest honor within their power to bestow.

His death occurred June 20th, 1898, at Pittsburg, Pa., where, on his way home from a meeting of the Mississippi River Commission, he stopped to visit Mr. Godfrey Stengel, a life-long friend who, forty-nine years before, had come with him on the same ship to America. He died very suddenly of acute heart failure while walking home from one of the parks in company with Mr. and Mrs. Stengel. Up to the last moment he was in excellent spirits and died without pain, as without fear.

As an engineer, Colonel Flad was remarkable for his great fertility of invention. For every new problem he had not only one, but many solutions, and the rapidity with which he grasped all its conditions and framed his plans to meet them amounted to genius. In doing this he was not limited by precedent, but looked instinctively for new and better methods than any before known. In boldness and origin-

ality he has had but few equals in the annals of the profession. And, like the most successful workers in every field, he delighted in his work for its own sake. Nothing could exceed the interest with which he attacked a new problem, and he gave himself no rest until he had solved it. The solution once found, however, the whole subject ceased to interest him, and he passed on to something new. This trait is illustrated by the fact that, although he took out numerous patents for new and useful inventions, to their introduction and utilization he gave no thought. It was the work, rather than its rewards, for which he cared.

As a man he was equally great. His unassuming modesty, his perfect candor and simplicity, his unflinching courage, his absolute fidelity to his convictions, his single-minded subordination of personal to the public welfare—qualities which were written in every line of his face and manifested in every act of his life—all stamped him as a man of the highest type. Anyone who knew him believed in him—without limit. His name was a synonym for fidelity and skill, and all knew that every work committed to his charge would be well done and come from his hand as sound and flawless as the man himself.

This evident and perfect integrity of purpose made his public service a legacy of incalculable value to his fellow citizens. His life was a demonstration of how honorable the public service can be made, and is an encouragement to those who have not yet lost faith in the possibility of having this service in all its branches lifted to the same standard to "abate no jot of heart or hope," but still work on for the accomplishment of this high end.

To his fellows in his chosen calling, his name and example are especially precious; for in him was realized the highest ideal of the engineer—a man of trained intellect, controlled by an iron will and directed to the noblest public ends. And his success in attaining this ideal will inspire others to frame their lives upon the same noble lines.

SIR CASIMIR STANISLAUS GZOWSKI, M. Am. Soc. C. E.*

DIED AUGUST 24TH, 1898.

Casimir Stanislaus Gzowski was born at St. Petersburg, Russia, March 5th, 1813. He was the son of Stanislaus, Count Gzowski, who held a commission in the Imperial Guard, and was descended from an old Polish family, ennobled in the sixteenth century. At the age of 9 years the son was placed in the Military Engineering School at Kremnitz, and in 1830 he received a commission in the engineering branch of the Imperial service. He, in company with many other officers of his nationality, was active in the Polish insurrection of 1830-31, in consequence of which he was exiled, after several months spent in confinement in a military prison. He came to the United States and landed in New York City in the summer of 1833, a stranger to the language spoken, without friends and with but little money.

His first task was to acquire a knowledge of English. He had a command of Continental languages, and while pursuing his studies he obtained the means of livelihood by teaching French, German and Italian and by giving lessons in drawing and fencing. Removing to Pittsfield, Mass., he began the study of law with Mr. Parker Hall. He was admitted to citizenship in the United States and to the bar in Pennsylvania in 1837. He continued the practice of law for four years and then went to Toronto, with a view of securing a contract in connection with the widening of the Welland Canal.

Here he met some of the leading public men of the Dominion at that time, and Sir Charles Bagot, then at the head of the Canadian Government, formed a high opinion of his ability, and procured for Mr. Gzowski an appointment in the Public Works Department. This was the beginning of his long and distinguished career in the Dominion of Canada. For six years he was Superintending Engineer of Roads and Harbors in Western Ontario, and for three years engineer of the harbor works at Montreal and consulting engineer of the ship canal improvement between Montreal and Quebec. He became naturalized as a British subject, and in 1848 left the Department of Public Works to become Chief Engineer on the construction of the St. Lawrence and Atlantic Railway, now a part of the Grand Trunk system. He resigned this position in 1852 to enter into partnership with Sir Alexander Galt, Mr. Luther H. Halton and the Hon. D. L. Macpherson, for the construction of the Grand Trunk Railway from Toronto to Sarnia, from Port Huron to Detroit, and from London to St. Mary's. In 1857 he established the Toronto Rolling Mills for the manufacture

*Prepared by the Secretary from information on file at the House of the Society.

of rails and other railway material. In 1870 he was appointed a member of a commission to inquire into the water communications of the Dominion, and in 1871 the firm of Gzowski & Macpherson constructed the International Bridge over the Niagara River. While in charge of this work, Mr. Gzowski was also called upon to report upon the enlargement of the Welland Canal and on the Baie Verte canal scheme. He was consulted frequently by the Dominion Government on matters relating to railways, canals and harbors.

Mr. Gzowski was chairman of the Niagara Falls Park Commission, one of the founders of the Canadian Society of Civil Engineers and its President for three consecutive years. He was deeply interested in military matters and in the promotion of international marksmanship contests. He rose to the rank of Colonel in the Canadian militia, and in 1879 was made an honorary aide-de-camp to the Queen. He held office in 1896, as Administrator of the Government of Ontario. In 1890, in recognition of valuable services rendered to the Dominion of Canada, he was created a Knight Commander of the Order of St. Michael and St. George.

Mr. Gzowski was elected a member of the American Society of Civil Engineers, December 2d, 1868, and served as one of its Directors during 1894.

FRANCIS RINECKER, M. Am. Soc. C. E.*

DIED APRIL 10TH, 1899.

Francis Rinecker, of Würzburg, Germany, was born December 16th, 1843. He studied mathematics and mechanical engineering under Professer F. Redtenbacher at the Polytechnic School at Carlsruhe, from 1858 to 1862, and civil engineering under Professor C. Culmann, at the Polytechnic School at Zurich, from 1862 to 1864. He began practical work in November, 1864, as assistant engineer of the Bavarian State Railways, C. Hohenner, chief engineer. He was engaged until November, 1867, in laying out and constructing part of the Munich and Ingolstadt Railway. In 1868 Mr. Rinecker traveled in Germany and France, and, coming to the United States, worked as an architectural draftsman from August of that year to April, 1869, in New York City, where he served with Charles Pfeifer, General Burger and J. Kellum. Turning to railway work in May, 1869, he was engaged successively as draftsman, assistant engineer and principal assistant of the construction department of the Pittsburg, Fort Wayne and Chicago Railway Company, and subsequently by the Pennsylvania Company, until June, 1873. Mr. Rinecker surveyed and constructed for the company first named the Erie and Pittsburg Railroad and the Ashtabula, Youngstown and Pittsburg Railroad, and conducted preliminary surveys for the Shenango Valley and Alliance Railroad, of which F. Slataper, M. Am. Soc. C. E., was chief engineer.

Returning to Europe, Mr. Rinecker served for two and a half years, from January, 1874, as Division Engineer of the Swiss Central Railway Company, for surveys and construction of the Liestal and Oensingen Railway and the Liestal and Waldenburg Railway (R. von Würthenau, Chief Engineer). From 1876 to 1883 he was engaged in literary pursuits with the especial view of introducing the rack on the St. Gothard and other mountain railways. He established in August, 1883, the firm of Rinecker, Abt & Co., at Würzburg, Germany, for the Abt system of mountain railways, and personally superintended the construction of the Lehesten & Oertelsbruch Railway and the narrow-gauge quarry tracks at Oertelsbruch and their extensions on the Abt combination system. From the establishment of this firm until his death, Mr. Rinecker had undertaken the construction of a number of Abt rack railways, including the Harzbahn in Germany, the Usui Toge Railway in Japan, the Transandine Railway, the Nilgiri Railway, the Mount Lyell Railway in Tasmania, the Mount Morgan Railway in Queensland and the Snowdon Railway.

* Memoir prepared from data furnished by Herr E. Wirsing, Würzburg, Germany.

In 1891–92 Mr. Rinecker traveled around the world, remaining in Cape Colony, New Zealand, Australia and Japan for several months. In these countries he made the acquaintance of many of the principal engineers, and studied the application of the Abt system to the conditions presented in each land. After his return he lived quietly at Würzburg, and died from influenza.

Mr. Rinecker was a member of the Institution of Civil Engineers, and was elected a Member of the American Society of Civil Engineers August 7th, 1872.

ROBERT DELOS ROWE, M. Am. Soc. C. E.*

DIED MARCH 14TH, 1899.

Robert Delos Rowe was born at Sheridan, Ill., August 24th, 1863, and received a common-school education, with an advanced course in the graded school of that place, under the tutelage of Professor F. Y. Hamilton. He went into the field as rodman on the construction of the Atchison, Topeka and Santa Fé Railroad in Kansas, in 1878, and served as inspector of bridging in New Mexico in 1879–80, narrowly escaping the Indians during the Victoria troubles of 1882. Mr. Rowe returned to school in 1883–84, and after taking an engineering course at Valparaiso, Ind., under Professor Bogart, was advanced as Resident Engineer of the New Mexico Division of the Atchison, Topeka and Santa Fé Railroad, which position he held until 1892. Removing then to Chicago, he built the South Chicago City Railway and operated it for about two years. He then engaged in private practice in the engineering profession, became junior member of the firm of Rowe & Rowe of Chicago, planned the large timber preserving works at Somerville, Tex., and also similar works for the Santa Fé Pacific Railroad at Bellemont, Ariz.

Mr. Rowe married in 1885 Miss Allie Kyger, of Mulberry, Ind., who died one year later. He married again in 1897 Miss Milla Y. Campbell, of Chicago, who survives him. His death resulted from an attack of pneumonia. Mr. Rowe was a man of high acquirements and strict integrity, and had earned the confidence of his superiors and the love of his associates and employees.

Mr. Rowe was elected a Member of the American Society of Civil Engineers January 2d, 1890.

* Memoir prepared by Samuel M. Rowe, M. Am. Soc. C. E.

WILLIAM ULYSSES SCOTT, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 26TH, 1898.

William Ulysses Scott was born in Hoboken, N. J., January 18th, 1870. From his schoolboy days he showed remarkable mathematical abilities, and these brought him into prominence in his studies after his removal with his parents to Chicago, Ill., which occurred when he was twelve years old.

His first engagement was in March, 1888, with the Wheeling and Lake Erie Railroad, on the preliminary survey between Bowerston and Martins Ferry, Ohio. He spent nearly two years in the service of this road, rising to the position of assistant in the Toledo office, where he was engaged in making calculations for preliminary and location estimates. In the winter of 1889–90 he was employed with John Haulon, of Canton, Ohio, on the preliminary survey for the Toledo and Walhonding Valley Railroad, and on the survey of the canal front at Massillon, Ohio. For two periods, in 1890 and 1891, he was employed as Assistant Engineer on the maintenance of way of the Second Division of the Illinois Central Railroad, and for three months preceding March, 1891, he was transitman on the location survey of the Fort Wayne, Terre Haute and Southwestern Railway.

In March, 1892, he again entered the employ of the Illinois Central Railroad Company as Assistant Engineer of the Louisiana Division. He had charge of a variety of engineering work, including the laying out of yards and company property, the construction of the Pacific Junction incline, the Poydras elevator, survey for the Stuyvesant dock terminal, and round houses at New Orleans, La., and Canton, Miss.

Mr. Scott had edited a book of logarithms, tables and short methods of computation, and was the author of numerous tables for frogs and switches, which, though never published, have been found of service by many engineers. He was also expert in laying out and personally supervising intricate track work.

He was modest and unassuming in manner, highly esteemed by the officials of the companies he served, and popular with those persons with whom his work brought him in contact.

Mr. Scott was elected an Associate Member of the American Society of Civil Engineers, February 3d, 1897.

* Memoir prepared by the Secretary from papers on file, and from information furnished by Hartwell P. Farrar, B. M. Harrod and W. W. Hayden, Members, Am. Soc. C. E.

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OF THE

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